Research Report
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LONG-TERM BENEFITS OF STABILIZING SOIL SUBGRADES

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KENTUCKY TRANSPORTATION CENTER
176 Raymond Building
University of Kentucky
Lexington, Kentucky 40506-0281

(859) 257-4513
(859) 257-1815 (FAX)
1-800-432-0719
www.ktc.uky.edu
ktc@engr.uky.edu

Research Report
KTC- 02-19/SPR-196-99-1F

Long-Term Benefits of Stabilizing Soil Subgrades

by

Tommy C. Hopkins
Program Manager and Chief Research Engineer

Tony L. Beckham
Research Geologist

Liechung Sun
Senior Research Engineer

Bixian Ni
Research Engineer

Barry Butcher
Research Investigator

Kentucky Transportation Center

College of Engineering
University of Kentucky

In cooperation with the
Kentucky Transportation Cabinet
The Commonwealth of Kentucky
and
Federal Highway Administration

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Commonwealth of Kentucky
Transportation Cabinet
Frankfort, Kentucky 40622

James C. Codell, III
Secretary of Transportation

Clifford C. Linkes, P.E.
Deputy Secretary

Paul E. Patton
Governor

December 5, 2002

Mr. Jose Sepulveda
Division Administrator
Federal Highway Administration
330 West Broadway,
Frankfort, Kentucky 40602-0536

SUBJECT: Implementation Letter: Final Report
"Long-Term Benefits of Stabilizing Soil Subgrades,"
Research Report KTC- 02-19/SPR-99-196-1F

Dear Mr. Sepulveda:

Over the last six decades, the Kentucky Transportation Cabinet (KYTC) has been very active and supportive of research efforts to examine new ways of improving pavement design and performance. As early as the late forties, pavement research in Kentucky focused on developing a pavement design system that was compatible with the soils and geology of Kentucky. Numerous field and laboratory research studies were performed at that time. Over the next three decades, the pavement design system gradually evolved and was modified on several occasions as traffic loads and volumes increased. In the early eighties, a mechanistic model (layered elastic model) was used to aid in the development of new pavement design curves. Those curves, which relate traffic loadings, CBR, and pavement thickness, have been used over the last two decades. More recently, KYTC has sponsored several research studies that will allow the use of new mechanistic pavement design models in a framework developed and sponsored by the American Association of Highway Transportation Officials (AASHTO).

During the early eighties, it became evident, however, that more attention should be focused on the poor engineering properties of Kentucky soils and their effects on pavement behavior and design. Difficulties were encountered in constructing pavements on weak soil subgrades because of an increase in construction traffic loads and volume. Most pavements in Kentucky are constructed on clay soils, which have poor engineering properties and very low bearing strengths when exposed to water. The problem was magnified when subgrades were left exposed over the winter. As subgrade soils absorb water and swell, soil density decreases and causes a loss of bearing strength. Difficulties were frequently encountered in attempts to construct pavements on the softened soils. Soils had to be dried and recompacted before pavements could be built. Research was sponsored by KYTC to examine ways of avoiding early construction problems.

Chemical stabilization was examined as one means of increasing subgrade strength. As shown by research in the mid eighties, when chemical admixtures are mixed with Kentucky soils the strengths of the soil-chemical mixtures are several times greater than the strengths of untreated soils. Based on recommendations by the Geotechnology Section of the University of Kentucky Transportation Center (UKTC), a major subgrade stabilization program using chemical admixtures was initiated in the late eighties. Short-term follow-up studies at selected sites showed that in situ strengths of the soil subgrade - chemical admixtures were several times greater than the untreated soil strengths. Pavement failures during construction were eliminated when chemical stabilization was used.

Moreover, chemical stabilization provided a good "working " platform for constructing the pavement and permitted the continuous flow of construction traffic at all times of the year. By increasing bearing strengths of subgrades, compaction of pavement layers was made much easier. In the short-term, chemical stabilization worked very well.

Since the inception of the chemical stabilization program, subgrades at more than 100 roadway sites have been stabilized. In the late nineties, KYTC decided that a comprehensive review of the chemical stabilization program was needed. In particular, questions concerning the longevity, durability, bearing strengths, structural credit, and economics of subgrades mixed with chemical admixtures, as well as the general performance of pavements resting on chemically treated subgrade soils, were to be examined. In-depth field and laboratory studies were conducted at twenty selected flexible pavement roadway sections. Chemical subgrade admixtures included hydrated lime, Portland cement, and two byproducts. The byproducts were lime kiln dust (LKD) and AFBC (Atmospheric Fluidized Bed 'Combustion) ash—a lime byproduct produced at a local oil refinery.

Findings reported herein showed that in situ CBR values of the chemically treated soil subgrades were 12 to 30 times greater than the in situ CBR values of the untreated subgrades. In all cases, chemical admixtures were highly effective in improving the bearing strengths of soil subgrades. Strength of treated subgrades contributed significantly to the structural integrity of the pavement. Proposed structural layer coefficients for subgrades mixed with hydrated lime, Portland cement, LKD, or a combination of hydrated lime and Portland cement were nearly equal to the structural layer coefficient for granular base. The proposed values were verified from actual field data where varying degrees of structural credit had been given to the chemically stabilized subgrades. Hence, "in service" structural layer coefficients were actually observed for sections of roadways containing treated subgrades that ranged in ages from about 12 to 15 years. Using the proposed values and assigning structural credit to treated subgrades in future pavement designs should pose no problem and will lead to cost savings developed in design.

Pavements resting on treated subgrades at the time of the study were rated "good" based on KYTC pavement condition criteria. Rideability indices of the pavements are generally very high. At two of the twenty roadway sites, asphalt concrete overlays were constructed after about 15 years. However, in both cases, the estimated accumulated ESAL values were about equal to the assumed design ESAL values—the lives of these pavement sections had expired. Economical analysis show that for the same structural number, or strength, pavements resting on chemically treated subgrades can be constructed at costs lower than pavements resting on untreated subgrades. Chemical stabilization is very economical.

Finally, moisture content data show that a "soft" zone, or layer, of soil generally exists at the top of untreated soil subgrades. In situ CBR values of this untreated zone generally ranged from about 1 to 5. However, this zone of weak material did not exist at the top of chemically stabilized subgrades—CBR values ranged from 24 to 59 at the 85th percentile test value. As noted herein, the existence of this soft zone at the top of untreated subgrades has major engineering implications. The strength of this material determines the pavement design thickness required to resist failure and adversely affects the future performance of the pavement. Chemical stabilization is very effective in relocating the soft layer from the top of the subgrade to the bottom of the treated layer. By positioning this soft layer at a greater depth, the stress applied to this weak zone is much smaller than the stress in untreated subgrades. Therefore, the potential damage that the soft layer can produce in the pavement is reduced, which provides a significant benefit to pavement performance. Chemical admixture stabilization is a good economical technique for improving subgrade strengths. KYTC will continue to use this method when circumstances dictate the need for this alternative stabilization method.

Sincerely,

J. M. Yowell

J. M. Yowell, PE
State Highway Engineer

JCC/JMY/MM/WLB/lg

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16. Abstract

Chemical admixtures have been used extensively since the mid-eighties in Kentucky to improve bearing strengths of soil subgrades. Most pavements in Kentucky are constructed on clayey soils. Although short-term observations at a small number of sites showed that chemical stabilization worked very well, a need existed to perform a more comprehensive review and to assess the long-term benefits of this subgrade stabilization method. The main intent of this study was an attempt to address questions concerning bearing strengths, longevity, durability, structural credit, economics, and performance of pavements resting on soil subgrades mixed with chemical admixtures. In-depth field and laboratory studies were performed at fourteen roadway sites containing twenty different treated subgrade sections. Ages of the sites range from about 8 to 15 years. About 455 borings were made at the various sites. Air, instead of water, was used as the drilling media. In-situ CBR tests were performed on the treated subgrades and the untreated subgrades lying directly below the treated layers. Index tests and resilient modulus tests were performed on samples collected from the treated and untreated subgrades. Falling weight deflectometer (FWD) tests were performed. At the 85th percentile test value, the in situ CBR values of subgrades mixed with hydrated lime, Portland cement, a combination of hydrated lime and Portland cement, and a byproduct (MKD) obtained in the production of hydrated lime were 12 to 30 times greater than in CBR values of the untreated subgrades. In-situ CBR values of the treated layer ranged from 24 to 59 while the in situ CBR of the untreated layer at the 85th percentile test value was only 2. Based on rating criteria of the Kentucky Transportation Cabinet, the conditions of the pavements at twelve sites could be rated "good" at the time of the study– pavement ages were 8 to 15 years-- and "good" at the end of the twenty-year design period, based on projected data. At two sites, thin asphalt overlays had been constructed after 15 years. However, accumulated values of ESAL at those sites had exceeded or were near the values of ESAL assumed in the pavement designs. At the 20th percentile test value, rutting depths of the pavements resting on the treated subgrades were less than about 0.27 inches. Structural layer coefficients, a_3, for use in pavement design of the different chemically stabilized subgrades were developed. The proposed values were verified at sites where reduced pavement thickness was used and "in service" structural coefficients could be observed. Back-calculated values of FWD modulus of the treated layers were about two times the values of modulus of the untreated subgrade. Resilient modulus of the treated subgrades was larger than the resilient modulus of the untreated subgrades. Moisture contents at the top of the untreated subgrade layers showed that a "soft" layer of material exists at the very top of the untreated subgrade. This soft zone did not exist at the top of the treated layer. This discovery has significant engineering implications. Future research will focus attention on an in-depth examination of this weak layer of soil. Chemical admixture stabilization is a good, durable and economical technique for improving subgrade strengths.

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TABLE OF CONTENTS

LIST OF FIGURES xi

LIST OF TABLES xv

EXECUTIVE SUMMARY xvii

INTRODUCTION 1

OBJECTIVES AND SCOPE OF STUDY 2

BACKGROUND 3

SURVEY OF SUBGRADE STABILIZATION PRACTICES IN THE UNITED STATES 9

SELECTION AND LOCATION OF STUDY SITES 10
Field Reconnaissance 10
Subgrade Stabilization Methods used in Kentucky 10
Locations and Attributes of Stabilized Highway Subgrade Sections 11
Coring Techniques and Field Testing Procedures 11

TESTING METHODS 13

SUBGRADE ATTRIBUTES AND FIELD TEST RESULTS 14
Anderson County, US 127 14
Boyle County, US 127, By Pass (NBL only) 15
Fayette County, US 25 17
Lee-Wolfe Counties, KY 11 18
McCreary County, US 27 22
Shelby County, KY 55 24
Hardin County, US 62 25
Owen County, US 127 28
Trigg County, US 68, West Bound Lane (only) 29
Boone County, KY 842 31
McCracken County, US 62 33
Hickman County, US 51 35
Breckinridge County, US 60 36
Daviess County, KY 331, River Port Access Road 38

INDEX PROPERTIES OF STABILIZED AND NONSTABILIZED SUBGRADES 39

RESILIENT MODULUS OF UNTREATED AND TREATED SOIL SUBGRADES... 39
Sampling 40
Testing Equipment 40
System Components 41
Test Data 41

ANALYSIS... 42
Comparisons of Index Properties of Stabilized and Non Stabilized Soils 42
Soil Classifications 42
Grain Sizes—Clay Fractions 42
In Situ CBR Values of Untreated and Treated Soil Subgrades 43
Determination of Structural Layer Coefficients, a_3, of Subgrades 44
Moisture Contents of Non Stabilized and Stabilized Soil Subgrades 44
Non-stabilized Subgrades 44
Stabilized Subgrades 45
Back Calculation of Structural Layer Coefficient, a_3 46
Using the 1993 AASHTO Design Guide 46
Using the Kentucky Design Curves 49
Resilient Modulus of Undisturbed Core Specimens from Stabilized and Non-Stabilized Subgrades 51
Review of Mathematical Models for Relating Resilient Modulus and Stresses 52
Comparisons of Resilient Modulus Models 54
Simple correlation analysis 54
Multiple correlation analysis 57
Results of Multiple Regression Analysis of Resilient Modulus Tests Performed on Untreated and Treated Subgrade Specimens 58
Falling Weight Deflectometer (FWD) Measurements and Back Calculations 61
Rutting Measurements of Pavement Sections 64
Evaluation of Pavement Conditions of Study Sections 65
Economical Analysis 68

SUMMARY AND CONCLUSIONS 74

ACKNOWLEDGEMENTS... 76

REFERENCES 76

APPENDIX A – Survey on the Usage of Chemical Admixtures in the United States for Stabilizing Highway Subgrades... 81
APPENDIX B – Global Positioning System (GPS) Locations of Pavement Borings Latitudes and Longitudes ... 93
APPENDIX C – Index Properties of Untreated Soils and Soils Mixed with Chemical Admixtures ... 101
APPENDIX D – Percent Finer than No. 10 US Sieve, No. 200 US Sieve, and 0.002-mm size Particles of Chemically Treated and Untreated Subgrades ... 107
APPENDIX E – In Situ Moisture Contents of Chemically Treated and Untreated Subgrades ... 111
APPENDIX F – Regression Plane Coefficients, k1, k2, and k3, obtained from Models 4 and 5, Equations 11 and 12. ... 121

LIST OF FIGURES

Figure 1. Survey response of states using chemical stabilization 9
Figure 2. Chemical admixtures used by different states 9
Figure 3. General locations of study sites 11
Figure 4. Performing in situ CBR tests 13
Figure 5. Pavement sections at the Anderson County site, US 127 14
Figure 6. Relationship of AADT and time..14
Figure 7. Accumulated ESALs as a function of time..14
Figure 8. Rideability index as a function of time 15
Figure 9. Percentile test value as a function of pavement rutting depth (in.).....................................15
Figure 10. Pavement section at the Boyle County site, US 127Bypass 15
Figure 11. AADT as function of time 16
Figure 12. ESALs as a function of time..16
Figure 13. Rideablity indices..16
Figure 14. Average rut depth as a function percentile test value 16
Figure 15. Pavement cross section at the Fayette County site, US 25 17
Figure 16. AADT as a function of time...18
Figure 17. Accumulated ESALs as a function of time..18
Figure 18. Average rut depth (inches)...18
Figure 19. In situ CBR values of kiln dust, hydrated lime, and cement and non-stabilized Subgrades of the KY 11 sections 19
Figure 20. In situ CBR values for the AFBC byproduct subgrade sections..19
Figure 21. Percentile test value versus moisture content at the top of the untreated subgrade...........20
Figure 22. AFBC cross sections..20
Figure 23. Soil-cement and untreated cross sections 21
Figure 24. Soil-hydrated lime-soil and untreated cross sections..21
Figure 25. Kiln Dust-Soil cross sections...21
Figure 26. AADT as function of time, KY 11 in Lee-Wolfe Counties 21
Figure 27. Accumulated values of ESAL as function of time in Lee-Wolfe Counties.......................21
Figure 28. Rideability Index as a function of time for KY 11 in Lee-Wolfe Counties.......................21
Figure 29. Relation between percentile test value and rut depth..22
Figure 30. Cross sections of US 27, McCreary County 22
Figure 31. AADT as a function of year. 23
Figure 32. Accumulated ESALs as a function of time..23
Figure 33. RI as function of time 23
Figure 34. Percentile test value as function of depth of rutting (in.)...23
Figure 35. Pavement cross sections...24
Figure 36. AADT as a function of time...24
Figure 37. Accumulated ESALs as a function of time..25
Figure 38. Rideability Index as a function of time..25
Figure 39. Percentile test value as function of depth of rutting...25
Figure 40. Pavement cross sections...26
Figure 41. Relationship between growth and time..27
Figure 42. Accumulated ESALs as a function of time..27
Figure 43. Rideability Index as a function of time for US 62 in Hardin County 27
Figure 44. Depth of rutting as a function of percentile test value 27
Figure 45. Pavement cross section 28

Figure 46. AADT as a function of time........28
Figure 47 Accumulated ESALs as a function of time........29
Figure 48. Rideablity index as a function of time........29
Figure 49. Rideablity index as a function of time........29
Figure 50. Cross section........29
Figure 51. AADT as a function of time........30
Figure 52. Accumulated ESALs as a function of time........30
Figure 53. Rideability indices........31
Figure 54. Depth of rutting........31
Figure 55. Cross Sections........31
Figure 56. AADT-time relationship........32
Figure 57. Value of ESALs as a function of year........32
Figure 58. Rideability index as a functiin of........33
Figure 59. Cross sections........33
Figure 60. AADT-time relationship........34
Figure 61. ESALs as a function of time........34
Figure 62. Trend of rideability index and time........34
Figure 63. Depth of rutting........34
Figure 64. Cross section........35
Figure 65. AADT as a function of years........35
Figure 66. Estimated accumulated ESALs as a function of years........36
Figure 67. RI as a function of years........36
Figure 68. Depth of rutting........36
Figure 69. Pavement cross sections of borings........36
Figure 70. AADT as a function of years........37
Figure 71. ESALs as a function of time........37
Figure 72. RI as a function of time........37
Figure 73. Depth of rutting........37
Figure 74. Cross sections........38
Figure 75. AADT as a function of time........39
Figure 76. Estimated accumulated ESALs........39
Figure 77. View of resilient modulus testing equipment........41
Figure 78. View of loading actuator........41
Figure 79. View of LVDTs mounted on the sides of the specimen inside the triaxial chamber........41
Figure 80. Percentile test value versus percent finer than 0.002 mm- particle size –hydrated lime........42
Figure 81. Percentile test value versus percent finer than 0.002 mm- particle size—cement........42
Figure 82. Percentile test value as a function of the in situ CBR of stabilized subgrades........43
Figure 83. Relationship between the coefficient, a_3, for different subbases at the AASHO Road Test (1960), and CBR........44
Figure 84. Soft zone of soil situated at the top of the soil subgrade (untreated section)........44
Figure 85. Comparison of moisture contents of specimens obtained from in situ CBR locations and resilient modulus specimens obtained below the top the untreated soil subgrades........45
Figure 86. Comparison of moisture contents of specimens obtained from in situ CBR locations at the tops of the stabilized layer and non-stabilized layer........45
Figure 87. Comparison of moisture contents of specimens obtained at all in the treated and untreated subgrades........46
Figure 88. Graphical User Interface (data entry screen) of a computer program for performing Back calculations to determine the layer coefficient, a_3, of the chemically treated soil subgrade (subbase)........47

Figure 89. Graphical User Interface for obtaining thickness of pavement components and structural number from the Kentucky Design Curves..49
Figure 90. Resilient modulus as a function of the sum of principle stresses, σ_{sum}............................54
Figure 91. Relationship between deviator stress, $\sigma_{d,}$ and resilient modulus, M_r 54
Figure 92. Relationship between deviator confining stress, $\sigma_{3,}$ and resilient modulus, M_r 54
Figure 93. Prediction of relationship between resilient modulus, M_r and confining stress, $\sigma_{3,}$ using Model 4 55
Figure 94. Prediction of relationship between resilient modulus, M_r and confining stress, $\sigma_{3,}$ using Model 3 55
Figure 95. Prediction of relationship between resilient modulus, M_r and deviator stress, $\sigma_{d,}$ from the new model 56
Figure 96. Prediction of relationship between resilient modulus, M_r and the sum of principle stresses, $\sigma_{sum,}$ from the new model 56
Figure 97. Prediction of relationship between resilient modulus, M_r and confining stress, $\sigma_{3,}$ from the new model 56
Figure 98. Percentile test value as a function of R^2 obtained for Models 1 through 5.......................57
Figure 99. Percentile test value as a function of R^2 obtained for Models 1 through 5.......................57
Figure 100. Percentile test value as a function of R^2 obtained for Models 1 through 5.....................57
Figure 101 Comparison of R^2 results of Models 4 and 5 58
Figure 102. Least square regression plane of Model 4...58
Figure 103. Least square regression plane of Model 5...58
Figure 104. View of the regression plane of Model 4 in the direction of M_r-σ_d 59
Figure 105. Examples of least square regression planes from Model 5 for soil-cement and untreated specimens 59
Figure 106. Percentile test value as a function of resilient modulus of untreated and soil-cement specimens 59
Figure 107. Percentile test value as function of resilient modulus of untreated field specimens and soil-hydrated lime field specimens..60
Figure 108. Percentile test value as a function of resilient modulus of untreated and soil-hydrated lime field specimens...60
Figure 109. Percentile test value for stabilized and non-stabilized subgrades as a function of modulus obtained from FWD measurements and calculations....................................61
Figure 110. Comparisons of average values of FWD of base aggregate, chemically stabilized subgrades, and non-stabilized subgrades..63
Figure 111. Average FWD modulus of DGA or crushed stone base as a function of the average FWD modulus of soil-cement and soil-hydrated lime subgrades 63
Figure 112. Relationship between structural layer coefficient, a_2, and modulus (after AASHTO 1993)...64
Figure 113. Relationship between resilient modulus and layer coefficient, a_i (after NCHRP Project 1-37A, 2002 Milestones)..64
Figure 114. Rideability index as a function of the average annual daily traffic and condition of pavement 65
Figure 115. Rideability index as a function of pay value used to reward contractors for constructing pavements with good ride qualities 68
Figure 116. Average rideability index and age for the twelve District Offices of the Kentucky Transportation Cabinet and the year 2000 68
Figure 117. Pavement scenarios used in the economical analysis 69

LIST OF TABLES

Table 1. Listing of Pavement Sections and Attributes 12
Table 2. Listing of field and laboratory tests........ 13
Table 3. US 127, Anderson County 14
Table 4. Boyle County, US 127 16
Table 5. Results of field tests at the US 25 site in Fayette County 17
Table 6. KY 11 subgrade experimental stabilization sections 19
Table 7. Listing of boring data obtained in the summer of 2000 at the KY 11 site in Lee and Wolfe Counties........ 20
Table 8. Listing of boring data obtained at the US 27 site, McCreary county........ 23
Table 9. Drilling results for Shelby County, KY 55 24
Table 10. US 62, Hardin County, Section 1, Eastbound lanes........ 26
Table 11. US 62, Hardin County, Section 2, Westbound lanes 26
Table 12. Drilling results of the Owen County site, US 127........ 28
Table 13. Summary of drilling results, Trigg County, US 68........ 30
Table 14. Drilling Results, Boone County, KY 842 32
Table 15. Drilling results, McCracken County, US 62 34
Table 16. Drilling results, Hickman County, US 51 35
Table 17. US-60, Breckinridge County........ 37
Table 18. KY 331, Daviess County........ 38
Table 19. In situ CBR values at the 85th percentile test value and structural layer coefficient of treated sections 43
Table 20. Summary of back calculated values of the AASHTO layer coefficient, a_3, of treated subgrades........ 49
Table 21. Summary of back calculated values of the Kentucky layer coefficient, a_3, of treated subgrades........ 50
Table 22. Summary of “in service” a_3 coefficients 51
Table 23 Correlation coefficients of models 3, 4, and 5 56
Table 24 Summary of R^2 values at the 90th percentage tests value obtained for the five models........ 57
Table 25. Average modulus values of study sections and pavement components obtained from FWD measurements and back calculations........ 62
Table 26 Average rutting measurements........ 65
Table 27 Summary of Rideability Indices and values of AADT of the study sections........ 66
Table 28 Summary of back-calculated values of the layer coefficients, a_3, of treated subgrades........ 67
Table 29. Unit cost used in the economical analysis of chemical stabilization. 71
Table 30 Summary of economical cost analysis 73

Table A-1. Survey results of the usage of highway subgrade stabilization in the United States 84
Table A-2. Survey results of the usage of highway subgrade stabilization in the United States 85
Table A-3. Survey results of the usage of highway subgrade stabilization in the United States 86
Table A-4. Survey results of the usage of highway subgrade stabilization in the United States 87
Table A-5. Survey results of the usage of highway subgrade stabilization in the United States 88
Table A-6. Survey results of the usage of highway subgrade stabilization in the United States 89
Table A-7. Survey results of the usage of highway subgrade stabilization in the United States 90
Table A-8. Survey results of the usage of highway subgrade stabilization in the United States 91

Table B-1. GPS positions of test sites 94
Table B-2. GPS positions of test sites 95
Table B-3. GPS positions of test sites 96
Table B-4. GPS positions of test sites 97
Table B-5. GPS positions of test sites 98
Table B-6. GPS positions of test sites 99
Table B-7. GPS positions of test sites 100

Table C-1. Index properties of stabilized and non--stabilized subgrades 102
Table C-2. Index properties of stabilized and non--stabilized subgrades 103
Table C-3. Index properties of stabilized and non--stabilized subgrades 104
Table C-4. Index properties of stabilized and non--stabilized subgrades 105

Table D-1. Results of grain-size analysis 108
Table D-2. Results of grain-size analysis 109
Table D-3. Results of grain-size analysis 110

Table E-1. Moisture contents of in situ CBR locations and resilient modulus specimens....112
Table E-2. Moisture contents of in situ CBR locations and resilient modulus specimens....112
Table E-3. Moisture contents of in situ CBR locations and resilient modulus specimens....113
Table E-4. Moisture contents of in situ CBR locations and resilient modulus specimens....113
Table E-5. Moisture contents of in situ CBR locations and resilient modulus specimens....114
Table E-6. Moisture contents of in situ CBR locations and resilient modulus specimens....114
Table E-7. Moisture contents of in situ CBR locations and resilient modulus specimens....115
Table E-8. Moisture contents of in situ CBR locations and resilient modulus specimens....116
Table E-9. Moisture contents of in situ CBR locations and resilient modulus specimens....116
Table E-10. Moisture contents of in situ CBR locations and resilient modulus specimens....117
Table E-11. Moisture contents of in situ CBR locations and resilient modulus specimens....117
Table E-12. Moisture contents of in situ CBR locations and resilient modulus specimens....118
Table E-13. Moisture contents of in situ CBR locations and resilient modulus specimens....118
Table E-14. Moisture contents of in situ CBR locations and resilient modulus specimens....119

Table F-1. Regression plane coefficients (k_1, k_2, and k_3) of Models 4 and 5 –soil-hydrated lime field specimens....122
Table F-1. Regression plane coefficients (k_1, k_2, and k_3) of Models 4 and 5 –soil-cement, lime kiln dust, and AFBC (Atmospheric Fluidized Bed Combustion ash) field specimens ..123

Table F-1. Regression plane coefficients (k_1, k_2, and k_3) of Models 4 and 5 –untreated field specimens 124

Table F-1. Regression plane coefficients (k_1, k_2, and k_3) of Models 4 and 5 –untreated field specimens 125

EXECUTIVE SUMMARY

In the mid-eighties, the Kentucky Transportation Cabinet (KYTC) initiated a major program to stabilize highway soil subgrades with chemical admixtures, which primarily were hydrated lime and Portland cement. This alternative form of subgrade stabilization was based on a recommendation by the University of Kentucky Transportation Center (UKTC) that showed that the low bearing strengths of subgrade soils in Kentucky needed improvement to avoid pavement failures during and after construction and that using chemical admixtures provided a good means of achieving this purpose. Although more than some 100- roadway sections have been treated chemically in the state since that time, there remained some lingering questions. What about the durability, bearing strengths, and longevity of subgrade soils treated with chemical admixtures? What about the performances of pavements resting on treated subgrades? Is chemical stabilization economical? Should chemically stabilized subgrades be given structural credit in determining the thickness of the flexible pavement during design? What structural credit should be assigned to the treated subgrade?

To address the many questions concerning chemical stabilization, a research study was initiated. The KYTC, in corroboration with UKTC, selected some fourteen roadways, which involved some twenty sections of soil subgrades treated with chemical stabilizers, for a detailed examination. Some 355 borings of the pavements at those sections were made and numerous in situ CBR tests were performed on the subgrades stabilized with chemical admixtures and untreated subgrades. More than 100 additional holes were bored at one roadway site (six sections) that preceded this study. A variety of laboratory tests were performed on samples of the treated and untreated subgrades. Tests included index tests, compaction, and resilient modulus. Falling Weight Deflector (FWD) tests were performed before coring on each section.

Significant findings and recommendations of this study are summarized as follows:

- Based on a survey, 26 states of 38 states responding to the survey used chemical admixtures to improve the bearing strengths of soil subgrades. All respondents noted that chemical stabilization was very beneficial. The most frequently used chemical admixtures were hydrated lime and Portland cement.
- Bearing strengths of subgrades stabilized with chemical admixtures, which ranged in ages from 8 to 15 years, were much larger than bearing strengths of untreated subgrades. Values at the 85th percentile test value of CBR of subgrades mixed with LKD (a byproduct produced in the manufacturing of hydrated lime), hydrated lime, a combination of first mixing with hydrated lime and then mixing with Portland cement, and Portland cement were 24, 27, 32, and 59, respectively. The CBR value of the untreated subgrade at the 85th percentile test value was only 2. Treated subgrade CBR values ranged from 12 to 30 times greater than CBR values of the untreated subgrade. The CBR value at the 85th percentile test value of an Atmospheric Fluidized Bed Combustion (AFBC) ash—a byproduct obtained in the production of oil-- was 9 at the 85th percentile test value. The CBR value of the soil-AFBC subgrade was about 4.5 times greater than the CBR value of the untreated subgrade. This study shows that chemically treated subgrades are very durable and long lasting.
- At four of the study sections, chemical admixtures (hydrated lime and Portland cement) were used to extract (or “dry”) excess water from the subgrade soils of those sites. This was performed in situ and provided a good alternate means of drying the soils so that pavement construction could proceed. Chemical admixtures react with water and the excess water is bound chemically with the admixture.
- The means for giving structural credit of chemically stabilized soil subgrades in the design of pavements was established and proposed in this study. Based on a relationship published by

AASHTO, which relates CBR and the structural layer coefficient, a_3, and using the CBR values of the stabilized subgrades at the 85th percentile test value, structural coefficients, a_3, of subgrades mixed with MKD, hydrated lime, hydrated lime-Portland cement combination, Portland cement, and AFBC were 0.10, 0.106, 0.11, 0.13, and 0.08, respectively. As a comparison, the value of the structural coefficient of granular base is generally accepted to be 0.14.

- Credible use of the proposed values of the a_3-structural layer coefficient cited above was established during this study at several sites using back-calculated coefficients based on the 1981 Kentucky Design curves. At four pavement sections, the "back calculated" or actual "in service" structural coefficients of soil-hydrated lime subgrades were 0.05, 0.09, 0.10, and 0.19. At three sections of soil-Portland Cement subgrades, the in service structural coefficients were 0.10, 0.16, and 0.18. At one section of soil-LKD subgrade, the in service coefficient was 0.10. At two other sections of soil-AFBC subgrades, in service coefficients were 0.09 and 0.15. Since the back-calculated structural layer coefficient was greater than zero, thickness of the pavement sections at these sites had been reduced and were smaller than the thicknesses that would have normally been required. Ages of the pavement sections ranged from 12 to 15 years. At the time of this study, none of these sections had asphalt overlays. Rideability indices of these sections at the time of this study ranged from 3.34 to 3.69. Projected RI-values (based on trend relationships of RI and time) at the end of twenty years range from 3.25 to 3.62. Based on pavement criteria (AADT as a function of RI) used by the KYTC in rating pavement condition, these reduced pavement sections resting on treated subgrades were rated good during the study. Based on projected RI-time relationships, the pavement conditions of the reduced sections at the end of a twenty–year period are generally rated good.
- Excluding the pavement sections described above, back-calculated values of the structural coefficient, a_3, of all sections ranged from about zero to minus 0.03. In those cases, no structural credit had been given to the stabilized subgrades in the pavement designs.
- Although two roadway sections containing subgrades treated with AFBC ash (and pavements) have performed very well over the last 15 years, this material should not be used except on an experimental basis. Initially, during construction, shortly after the asphalt bases courses had been placed, and after a long period of rainfall, the pavements resting on the subgrades mixed with the AFCBC ash swelled and formed humps, which ran perpendicular to centerline. In depth research showed that swelling was caused by reactions that occur when the sulfates and sulfites in the ash are exposed to water. Based on detailed field measurements of swell, projections indicated that swell would essentially decrease to very small values. The base courses of the two AFBC sections were milled and the final asphalt surface layer was constructed. The sections have performed very well since that time in 1987.
- Moisture content data show that a soft layer of soil frequently exists at the top of untreated subgrades. On the basis of percentile test value, moisture contents measured at the very top of untreated subgrades were some 3-4 percent larger than moisture contents measured at points below the top of the subgrades. This is a significant finding and has major engineering implications.
- Data collected during this study showed that the in situ CBR, which is performed at the top of the untreated subgrade, is very small. At the 85th percentile test value, the CBR value of the untreated layers at all study sections was only 2. Values of CBR of this magnitude are normally expected for saturated soils. Past research shows that CBR values of unsaturated and "as compacted" soils normally range from approximately 10 to 40. Since the pavement

thickness is based on the CBR strength of the soil in a saturated state, then smaller values of CBR increase required pavement thickness. Hence, elimination, or minimizing the effect of this zone on the performance of the pavement has great engineering significance. However, the elimination of this soft zone when base aggregate is placed directly on the untreated subgrade would be difficult. Although granular bases function to eliminate water from the pavement, the base aggregate cannot prevent the development of a "soft zone" of soil at the top of the untreated subgrade because water flows downward, as well as lateral (provided the subgrade slopes). Consequently, the top portion of the untreated subgrade becomes saturated. When clayey soils are involved, which is generally the case in Kentucky, the top of the subgrade soil swells and loses bearing strength.

- The effects of this soft zone on pavement performance can be minimized when thick pavements are used. However, this is an expensive way to mitigate the effects of the soft zone. In this case, stress increases, induced by traffic stresses, in the soft zone are relatively small when compared to stresses that occur in the soft zone when thin pavements are used. If the pavement is very thin, then large deflections may occur in the soft zone and cause pavement cracking.
- The most economical means of mitigating the effects of the soft zone is to use chemical stabilization. Data collected in this study show that chemical stabilization does not remove the soft zone. However, when stabilization is used, the soft zone of soil occurs below the stabilized subgrade, and at greater depths then when stabilization is not used. The soft zone of soil did not exist at the top of the treated subgrades. CBR values at the 85^{th} percentile test value measured at the top the stabilized subgrades of all sites ranged from 24 to 59 (excluding the AFBC sites). At the AFBC sites, the value was 9, the minimum design value generally recommended for the subgrade. Hence, the effects of the soft zone on pavement performance are mitigated because stress increases, induced by wheel stresses, are much smaller at the bottom of the treated layer than at the top of the treated layer, or untreated layer. Moreover, chemically treated soils possess large cohesive strengths that allow the treated material to withstand large excess pore pressures that build up from traffic stresses and minimizes "subgrade pumping".
- Considering that the ages of the sites ranged from about 8 to 15 years, the rutting depths were generally considered to be small. At the 20^{th} percentile test value, the rutting depths were less than 0.27 inches.
- Chemical stabilization substantially increased the elastic modulus of untreated soils at all sites. Back-calculated values of modulus obtained from Falling weight deflector (FWD) tests of subgrades mixed with chemical admixtures are about two times greater than the back-calculated values of modulus of untreated soils. As the modulus of the stabilized subgrade increases, the modulus of the granular base increases and the structural layer coefficient of the base increases. Consequently, the structural number, SN, of the pavement increases.
- Chemical stabilization represents a very economical means of improving the poor engineering strengths of Kentucky soils. Based on structural number, SN, required by the 1981 Kentucky flexible pavement design curves, the costs of pavement sections constructed on stabilized soil subgrades are less than equivalent pavement sections constructed on non-stabilized soil subgrades. Moreover, the thickness of a pavement resting on a treated subgrade can be thinner than the thickness of a pavement resting on an untreated subgrade. For a flexible pavement measuring 36 feet in width, the average cost savings for soil-hydrated lime- and soil-cement subgrade stabilization was 19,100 dollars per mile.

INTRODUCTION

Most pavements in Kentucky are constructed on fine-grained, clays and silts. Some 85 percent of soils in Kentucky consist of clays and silts. The majority of highway subgrades are constructed with clays. When first compacted, these clayey soils usually have sizeable bearing strengths. As shown by past research (Hopkins 1970, 1991, and Hopkins and Beckham 1995), CBR strengths of soil subgrades immediately after compaction, typically, range from 15 to 40. However, shortly after the pavement is placed and the clayey subgrade is exposed to moisture, past research shows that CBR strengths decrease to a range of about 1 to 5. Obviously, low CBR strengths can affect pavement performances. Past studies show that low bearing strengths can cause premature failures of pavements and point to the need to stabilize soil subgrades.

If pavements are constructed immediately after the compaction on the clayey soils, then major difficulties are normally not encountered in placing and compacting layers of paving materials. Problems may arise, however, when surface and subsurface water penetrates the compacted clayey subgrades. Water from rainfall, snowmelt, and groundwater seepage enters the clayey subgrades, causes swelling, and produces a loss of bearing strength. The most susceptible, adverse period occurs when the subgrade has been exposed to the wetting conditions of winter and early spring. During this period, before paving, rutting may quickly develop in the softened subgrade and slow, or even halt, the movement of construction traffic. Because of a lack of a firm foundation, difficulties arise when attempts are made to compact the first lifts of pavement. When these situations develop, delays occur which require costly remedial measures. When subgrades lose bearing strength during construction, the subgrade must be reworked, or recompacted, before pavements can be constructed--a costly procedure.

Even when the construction of the pavement is successful, the bearing strength decreases significantly with the passage of time and exposure to moisture; this adversely affects the behavior of the pavement. Problems, as noted by construction and geotechnical engineers, frequently include the shoving and pushing of clayey subgrades under construction traffic, the lack of a firm working platform for constructing and compacting base and paving materials, and a loss of bearing strength during and after construction. Pavement subgrades must be stable during construction and perform throughout the design life of the pavement. Often, the subgrade is the weakest member of the pavement structure and is an important factor influencing pavement performance. The subgrade must be sufficiently stable during construction to prevent rutting, pushing, and shoving. The subgrade must also provide a sound platform so that the various pavement layers can be effectively and efficiently placed and compacted. The subgrade must serve as a "working platform," and possess strength so that large permanent deformations do not accumulate over a long period of time and affect the performance of the pavement. Pavement construction problems may be classified as follows:

- failures of weak soil subgrades under construction traffic loadings;
- failures of granular base courses under construction traffic loadings;
- failures of partially completed pavement/base materials under construction traffic loadings;
- premature failures of pavements shortly after construction; and
- difficulties in achieving proper compaction of granular base and pavement materials due to inadequate bearing strength of the soil subgrade.

In the mid-eighties, the Kentucky Transportation Cabinet began a major subgrade stabilization program. The stabilization program was initiated as a result of private communication (Hopkins 1987) with resident engineers of KYTC and research findings from soil subgrade research studies

(Hopkins and Sharpe 1985, Hopkins and Allen 1986, Hopkins 1987, 1991 and Hopkins et al 1988) conducted by the University of Kentucky Transportation Center. During discussions in 1984, resident engineers noted that they frequently encountered problems constructing pavements on subgrades that had been exposed during the winter season or rainy periods. They noted that the subgrade soils were usually very soft and had to be reworked before pavement construction could progress. In particular, they requested that a study be initiated to examine ways to prevent, or mitigate, this problem. The need to stabilize subgrades also developed as a result of several pavement failures experienced by the Cabinet during construction and shortly after construction in past years. Suggestions were made to the Kentucky Transportation Cabinet pavement designers to increase the structural integrity of subgrades by using chemical admixtures. Overall pavement thickness can be reduced in some cases and pavement life extended when the subgrade is stabilized.

Pavements constructed on stabilized subgrades should last longer than those constructed on untreated subgrades under equal traffic loadings. Information needs to be obtained to determine if the additional cost of stabilizing subgrades prolongs the life of the pavement. Undocumented and informal observations strongly indicate that pavements placed on stabilized subgrades outperform pavements that are placed on untreated subgrades. Moreover, past studies indicate that using mechanical compaction of soil subgrades is not, necessarily, sufficient to prevent premature failures of pavements. Past observations since 1987 indicate that chemically stabilized subgrades are much stronger than untreated subgrades. Moreover, sufficient data (Hopkins 1987, 1991 and Hopkins and Beckham 1995) shows that the swelling of clayey subgrades is prevented when the clayey subgrade is treated with hydrated lime, or Portland cement. By preventing swelling, it appears that the strengths of the subgrade remain very large throughout the life of the pavement.

OBJECTIVES AND SCOPE OF STUDY

The major objective of this study was to examine the long-term benefits of constructing pavements on stabilized soil subgrades. Soil subgrades stabilized with chemical admixtures, such as cement, hydrated lime, and various byproducts were examined and the pavement performances noted.

Many immediate benefits are obtained from subgrade stabilization, especially chemical admixture stabilization. For example, by improving the bearing strength and stiffness of the subgrade, a good working platform is established for supporting construction traffic and for compacting paving materials. Subgrade soils that have poor engineering properties may be used effectively when chemical stabilization is used. Therefore, construction can continue efficiently. From a long-term aspect, the use of chemical stabilization appears to increase the long-term cohesive strength of the subgrade. This large cohesive strength of the subgrade tends to resist large excess pore pressures in the subgrade caused by large vehicular traffic stresses.

Although short-term benefits of subgrade stabilization are readily apparent, more information regarding long-term benefits is needed. Before 1987, only a few chemically treated subgrade stabilization projects were constructed in Kentucky, although many subgrades were stabilized by mechanical means. For example, when chemical admixture stabilization is used, a question arises concerning the durability and longevity of the treated subgrade. However, well-documented, published case studies are difficult to locate. Since 1987, several chemical and mechanical stabilization projects have been built. Major aims of this study are to examine several selected subgrade stabilization projects in more detail and consolidate information so that long-term benefits may be documented and evaluated. This study will focus on the long-term benefits of chemical stabilization. A major aim of this study was to examine the long-term durability of soil subgrades treated with chemical admixtures. Another task was to establish the means for assigning structural credit during design to subgrades mixed with chemical admixtures.

BACKGROUND

Observed differences (Hopkins 1991, Hopkins et al 1995, Hopkins and Beckham 2000) between pavement design assumptions and actuality -- the actual conditions faced by the field construction engineer -- have led to several pavement construction problems in past years. Pavement problems, or premature pavement problems, have occurred after construction. As a sampling of those construction problems, from about May 1986 to November 1989 -- about 3.5 years -- the Geotechnical Branch (*private communication)*[1] of the Kentucky Transportation Cabinet was involved in developing contingent and remedial plans at more than 40 highway construction sites. Personnel of the University of Kentucky Transportation Center were involved in some of the pavement failures.

Pavements are typically designed to support anticipated traffic loadings after the total pavement system is constructed. Usually, no consideration is given to the need to support heavily loaded vehicles, such as gravel or concrete trucks, during construction. It is assumed that pavements can be constructed as designed. The question of constructability is frequently overlooked and left to the field and geotechnical engineers to confront (Hopkins and Sharpe 1985, Hopkins 1991, 1994a, 1994b, and Hopkins et al 1994a, b). A common assumption is made that if the soil subgrade is compacted to 95 percent of standard (AASHO T 99) maximum dry density, and ± 2 percent of optimum moisture content, then construction of the pavement, as designed, should not present a problem. That is, if proper compaction is obtained, then the bearing strength, of the soil subgrade is sufficient to withstand construction traffic loadings. Compaction of soil subgrades is an essential element in the construction of pavements. This assumption fails to recognize that subgrade strength and stability varies during construction and throughout the life of the pavement and that subgrades, when constructed of weak soils, may not have adequate bearing strength to withstand construction traffic loadings. Damaged subgrades and partially completed pavements during construction may also lead to poor performance of the pavement after construction.

Past research (Hopkins 1991, 1994a,b, Hopkins and Allen 1986, Hopkins et al 1988, 1994a,b, 1995) conducted since 1987 helped establish a major highway subgrade stabilization program in Kentucky. To establish and implement a subgrade stabilization policy and program, many issues had to be considered and resolved. Some of the important issues, as listed and discussed in the earlier works, were as follows:

- Factors that affect and influence the short-and long-term behaviors of untreated subgrades.
- Minimum subgrade strength required to sustain construction traffic loadings and prevent bearing capacity failures of the subgrade.
- Use of laboratory strengths to predict long-term field strength of subgrades.
- Method of selecting design strengths of untreated and treated subgrades.
- Types of stabilization methods.
- Method of determining the optimum percentage of a chemical admixture when chemical stabilization is used.
- Treatment depth required to sustain construction traffic loadings when chemical admixture stabilization is used.

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[1] Private communication with Doug Smith, former construction liaison, and Henry Mathis, former Branch Manager, respectively, of the Geotechnical Branch, Division of Materials, of the Kentucky Transportation Cabinet.

- Comparison of the long-term strengths of treated subgrades to the long-term strengths of untreated subgrades.
- The effect of wetting-drying behavior on strengths of untreated and chemically stabilized subgrades.
- Longevity of subgrades treated with hydrated lime and cement.
- Rapid methods for the assessment of the overall bearing strengths of untreated and treated subgrades.
- General performances of flexible pavements constructed on chemically treated subgrades and the potential for reducing maintenance.
- Cost and economical benefits of chemical admixture stabilization.
- Soil subgrade conditions where hydrated lime and cement should not be used.
- Long-term benefits of stabilization.
- Resilient modulus of chemically treated soil subgrades (and the resilient modulus of untreated soil subgrades).

Some factors that significantly affect the behavior and performance of highway pavements and subgrades include the geologic setting and soil types existing at a given highway site. Physical properties of the subgrades, such as compaction degree, swelling tendencies, and the presence of moisture, may also affect the behavior and performance. Types of soils available at a given location in Kentucky for constructing subgrades are controlled by site geology since major portions of Kentucky's soils are residual -- soils that are the result of the weathering of bedrock. For example, soils derived from clayey shales, such as the Kope Geological Unit, in the northern regions of Kentucky, have very poor engineering properties (Hopkins and Deen 1983). Pavements placed on subgrades constructed with these types of soils have notoriously performed poorly. In comparison, pavements constructed on soils derived from the New Albany Geologic Unit have generally done very well. Moreover, subgrades constructed with New Albany Shales appear to perform reasonably well (Hopkins[2], Hopkins and Beckham 1995, and Hopkins et al 1991). Statistically, about 85 percent of Kentucky soils consist of clay and silt -- materials that have poor engineering properties.

Although compaction of clayey soils increases shear strength, compaction alone will not, necessarily, insure that a subgrade will act properly throughout pavement life. Subgrades are subjected to the infiltration of water from surface runoff and subsurface seepage. Compacted clayey subgrades absorb water and swell. As swelling occurs, a loss of bearing strength occurs. Both field and laboratory data obtained from past research studies (Hopkins et al 1988, 1994a,b, 1995, and Hopkins 1991, 1994 a,b) illustrate this condition. Moreover, the use of drainage measures, although desirable, will not prevent the development of this situation because the subgrade will be exposed to water during some period of the pavement's life. Therefore, compaction and drainage measures used alone will not totally insure good performance of clayey subgrades and pavements.

When should subgrade modification be considered? To resolve this question, a bearing capacity model (Hopkins 1986, 1991, 1994a, b, 1995, and Hopkins and Slepak 1998, Slepak and Hopkins 1993, 1995a, b) based on limiting equilibrium was developed and used to analyze this problem. Relationships between undrained shear strength (and California Bearing Ratio -- CBR) of the subgrade and different tire ground contact stresses were developed for different factors of safety against failure. Therefore, if the tire contact stresses that may exist on the clay subgrade during

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[2] Private communication with the consulting engineer responsible for developing subgrade specifications for Section 20 of the Ashland-Alexendria (AA) Highway. It was suggested that a 2-foot thick layer of durable shale (slake durability index equals about 98 percent) be used as the subgrade. Measured values of in situ CBR of the durable shale subgrade over the last several years have generally exceeded 10.

construction are known, then the minimum strength necessary to sustain construction traffic may be found from the relationships developed from the past research studies. Using these relationships, engineers of the Kentucky Transportation Cabinet can rapidly detect difficulties during construction of the pavement layers or determine if the untreated or treated subgrade may fail under construction traffic. For example, if the anticipated tire stress is 80 psi (552 kPa), then the minimum in-situ CBR strength required to maintain incipient failure (factor of safety equals one) is about 6.5 (Hopkins 1991). However, to maintain good stability, the in-situ CBR strength should be about 9 or greater (factor of safety equal to 1.5). Minimum strengths required when the tire contact stress is some value other than 80 psi may be obtained from relationships shown by the past studies. The analyses showed that the in-situ CBR strength of the subgrade should be about 9 or 10 to avoid failure during construction of the first lifts of a pavement.

Using the above guideline, if subgrade modification is deemed necessary, then several techniques may be used to improve bearing strength. These methods can be broadly classified into two categories: mechanical and chemical. Mechanical methods include such traditional approaches as: controlling subgrade density-moisture, undercutting poor materials and backfilling with granular materials, proof rolling and re-rolling of the subgrade, mixing of stone aggregate with the clayey subgrade, using granular layers, and using granular layers reinforced with geofabrics. Detailed laboratory examinations of the technique of mixing stone aggregate into the soil subgrade have been conducted (Hopkins et al 1995 and Hopkins and Beckham 2000). As shown in those studies, a significant decrease in bearing strength occurs when the clay content (percent finer than the 0.002 mm-particle size) of the soil-aggregate mixture is greater than about 15. This stabilization technique is ineffective in mixtures containing large clay contents and exposed to moisture. According to KYTC personnel, this technique has performed poorly in the field and is no longer used.

The use of geofabrics, such as geogrids, to reinforce subgrades and improve bearing capacity of granular bases, was also examined (Hopkins and Beckham 1995) using a newly developed, (preliminary) version of the bearing capacity model (Slepak and Hopkins, 1993, 1995a, and 1995b). Results of these analyses show that the factor of safety increases some 10 to 25 percent when geogrids are used (Hopkins and Slepak 2002). However, stability analyses of field case studies need to be performed to confirm this result and to verify the reasonableness of the newly developed stability model. Moreover, future research needs to be performed to expand the capabilities of this model approach**.**

Chemical stabilization was a major focus of the reports (Hopkins et al) published in June 1991 and January 1995. Before 1987, chemical stabilization was used sparingly in Kentucky. Commercial chemical stabilizers include hydrated lime and cement. Only four sites, constructed before 1987, were found that used cement as the subgrade chemical admixture (Hopkins et al 1994a,b, 1995). No sites constructed before 1987 were found that used hydrated lime as the chemical admixture. Apparently, the first sites -- KY 11 and Section 19 of the Alexandria - Ashland Highway-- in Kentucky using hydrated lime as a subgrade stabilizer originated from research studies performed by University of Kentucky Transportation Center and the Kentucky Transportation Cabinet. Experimental sites, established in earlier studies, have been monitored for several years. In situ CBR strengths of the soil-hydrated lime subgrades, as well as untreated subgrades, have been measured in the experimental sections. The soil-cement subgrades (Hopkins et al 1994a and b) at the four old sites, which ranged in ages from about 9 to 38 years, are extremely stiff. In situ CBR strengths generally exceed 90. Flexible pavements constructed on the soil-cement subgrades generally have performed very well. Average history of the thin overlays is about 12-14 years for different locations on the different stretches of roadways.

Two byproducts were used at the KY 11 site near Beattyville, Kentucky (Hopkins et al 1988; Hopkins and Beckham 1993c, Hunsucker et al 1993a,b, and Hopkins and Beckham 1995). Two subgrade sections of this reconstructed route were treated with an Atmospheric Fluidized Bed

Combustion (AFBC) spent-lime (or any flue gas desulfurization material, Hopkins et al 1993a). Laboratory tests showed that the addition of the spent lime significantly increased the bearing strength. However, about two months after placement of the asphalt base layers, and after a rainy period, pavement buckling occurred at several locations. Swell data from standard CBR laboratory tests performed on the AFBC-soil mixtures did not indicate that swelling was a problem. As shown by subsequent tests, a long time period of delay occurred before swelling commenced. Based on laboratory swell tests, a theoretical estimate of the time for completion of primary swelling of the subgrade was made. Final surfacing, after pavement milling of buckled locations, was placed after the estimated time. After about 7 years, in situ monitoring showed that CBR strengths generally exceed 9 and rutting is less than about 7.6 mm (0.3 in.). To determine the causes of the swelling, subgrade specimens were obtained. X-ray diffraction (XRD) and scanning electron microscopy analyses were performed on the collected specimens. Analysis showed that the swelling behavior of the AFBC-treated subgrade was caused by the formation of ettringite and anhydrite gypsum-- types of minerals. Formation of these minerals and swelling appear to be closely related to the presence of calcium sulfate and sulfite. The recommendation was made to engineers of the Kentucky Transportation Cabinet that FBC byproducts should not be used as chemical admixtures in soil subgrades unless it could be shown that the long-term swelling, as determined from long-term laboratory swelling tests, of the FBC material is less than about 4 percent and the CBR strength is greater than above nine after the total swelling has occurred. Other work performed by the University of Kentucky Transportation Center (Hopkins et al 1993a, Hopkins and Beckham 1995) on FBC-type byproducts that contain significant amounts of sulfates confirms earlier observations and findings.

A second byproduct, lime kiln dust (LKD), was also used to treat a subgrade section of KY 11 (Hopkins et al 1988; Hunsucker et al 1993, and Hopkins et al Beckham 1995). After 7 years, the in situ CBR strength of the LKD-treated subgrade generally exceeds 90. Rutting of the pavement after 7 years is less than 0.25 cm (0.1 in.). Because of the superior performance of this pavement section, it was recommended that this byproduct could be used as a chemical admixture.

In situ CBR tests were performed at two highway routes over a period of about five years to determine if soaked, laboratory strengths represent long-term, field strengths,. The laboratory and field CBR values were graphed as a function of percentile test values; the laboratory strengths seem representative of field strengths. Therefore, it was recommended (Hopkins, June 1995) that soaked laboratory strengths could be used to select appropriate design strength of untreated clayey subgrades. Although this has been done in the past, data to support this design approach was obtained in an attempt to justify using soaked laboratory strengths.

When should soil subgrade stabilization be considered? Guidelines (Hopkins 1991,1995) for deciding when subgrade stabilization is needed were formulated and recommended to engineers of KYTC. If the CBR strength of a subgrade is below about 6.5, and the tire contact stress is 552 kPa (80 psi), then subgrade stabilization, such as chemical stabilization with hydrated lime or cement, should be considered. This important principle was established from results obtained from the newly developed bearing capacity model described in the report cited above. Cabinet engineers generally observe this recommendation. Based on the mathematical modeling (Hopkins 1991), interim design (memorandum) guidelines were issued (Hopkins and Hunsucker 1990).

If chemical stabilization is used, then two major questions arise: should the treated subgrade be considered merely as a construction, or working platform, or should it be considered a part of the pavement structure? How thick should the treated subgrade be to avoid failures during construction? To address the first question, core specimens were obtained at several highway sites from cement- and hydrated lime-treated subgrades. The specimens were obtained at the end of a 7-day curing period. Unconfined compression tests were performed on those specimens. Also, laboratory specimens were compacted and unconfined compression tests were performed on those specimens.

The compacted specimens had been aged for 7 days before testing. Results from laboratory and field unconfined compression tests were graphed as a function of percentile test values. Based on the 90th percentile test value, it was recommended that reasonable undrained design strengths for soil-cement and soil-hydrated lime subgrades were 711 kPa and 331 kPa (103 and 48 psi), respectively. These values correspond to CBR values of about 25 and 12, respectively. Values of dynamic modulus of elasticity are about 297,487 kPa (43,114 psi) and 152,594 kPa (22,115 psi), respectively. By using these values, at least part of the subgrade strength gain may be used in design. Presently, the Cabinet has adopted this approach, although, as we understand, the lower value of 152,594 kPa (22,115 psi) is being used for both soil-cement and soil-hydrated lime subgrades. Nevertheless, this idea has been implemented.

A design chart relating the required thickness for soil-cement and hydrated-lime to the CBR strength of the untreated subgrade found below the treated layers was developed using the newly developed, bearing capacity model (Hopkins, June 1991). A factor of safety of 1.5 and the undrained strength (or CBR) occurring at the 90th percentile test value (listed above) were used in those analyses.

During earlier studies (Hopkins et al 1986), a laboratory procedure (Hopkins and Beckham, 1993b) was developed for determining the optimum percentage of a chemical admixture that should be specified on a given project and for a given type of soil. Unique laboratory compaction equipment was designed and constructed. Working drawings of this equipment were transferred to the Geotechnical Branch of the Kentucky Transportation Cabinet. This procedure, including mathematical algorithms and a PC computer program for performing the necessary calculations to remold specimens, was adopted by the Geotechnical Branch and has been used routinely. In the procedure, the unconfined compression test is used to determine the optimum percentage of chemical admixture. After using the procedure for several years, engineers of KYTC decided that 5 percent (by dry mass) of hydrated lime was generally sufficient to stabilize most Kentucky soils. For this reason, the procedure is not always performed and 5 percent of hydrated lime is usually specified.

What method should be used in selecting the design strength of untreated and chemically treated soil subgrades? An in-depth analysis of several approaches to this problem was made; two case studies (Hopkins and Beckham, July 1994a and b) involving pavement failures were analyzed using a newly developed bearing capacity model (Hopkins 1991). The case studies were very useful in establishing the most appropriate method for selecting the design strength of a soil subgrade. It was recommended that KYTC engineers adopt a least-cost approach--proposed by Yoder (1969) and Yoder and Witczak 1975). This approach involves graphing the strengths (for example, CBR) as a function of percentile test values. If the cost ratio -- the unit maintenance cost to the unit initial cost -- is known or assumed, then the design percentile test value may be selected. Once this value is known, then the design strength is obtained. If the cost ratio is unknown, then the value of strength occurring at the 80th to 90th percentile test value may be selected for design purposes. It was shown that this is a good approach, as illustrated by the analyses of two case studies involving failures of pavements during construction. To implement and facilitate the use of this approach, a PC® (personal computer) computer program was developed for the Cabinet's engineers. The geotechnical staff of KYTC received training on the use of this program in earlier years.

In situ moisture contents and field CBR values of clayey subgrades at two experimental highway routes were monitored over a period of about five years (Hopkins et al 1995). A dramatic reduction in strengths of untreated clayey subgrades occurred with increases in moisture content and time. Such large decreases in strength must be considered in the design of pavements. Soaked laboratory strengths have been and are being used for predicting long-term field strengths. However, soaked strength from a laboratory test may not represent long-term field strength. This research study attempted to address that issue. Results obtained at two sites over a period of five years showed that the field CBR strengths were close to soaked laboratory CBR strengths.

Previous published case studies show that when soils contain high levels of soluble sulfates, large magnitudes of swelling may occur when hydrated lime or cement is used as chemical admixtures. Swelling of the treated subgrade adversely effects the pavement, that is, the pavement is prone to heave, or form "humps" that run perpendicular to the centerline. This condition (Hopkins et al 1993a, Hopkins et al 1988, and Hopkins et a1 995) may also occur if the chemical admixtures (byproducts) contains high levels of soluble sulfates. For example, FGD (Flue Gas Desulfurization) byproducts produced from coal-fired power plants contain high levels of soluble sulfates. Those materials also contain calcium oxide (quicklime), or calcium hydroxide, which reacts with clayey soils when mixed and increases shear strength. In either case, five conditions must exist to initiate swelling. These are as follows:

- High pH conditions,
- Adequate supply of alumina, silica, and carbonates -- sufficient clayey mineral content,
- Presence of sulfates (either in the soil or FGD byproduct),
- Correct temperature conditions
- Availability of water.

When these conditions exist, the formation of the minerals, gypsum, ettringite and thaumasite, occurs and the treated subgrade will swell. To date, no cases of pavement heave have been reported in Kentucky at sites where subgrades have been treated chemically with quick, or hydrated lime. Swelling did occur on two sections of KY 11. However, high levels of soluble sulfates were present in the FGD byproduct admixture and not in the soils. Other subgrade sections on this route were treated with hydrated-lime and cement. No swelling occurred. Although no cases of pavement swelling have been reported to date, using hydrated lime and cement as chemical subgrade admixtures in certain geological regions of Kentucky could potentially cause swelling problems. For example, the residual soils of the New Albany Geologic Unit have the potential to cause swelling problems. This unit contains pyrite, which is high in sulfur content. Identifying soils high in sulfate content was beyond the scope of this study. Additional research is needed for identifying suspect areas. Moreover, the use of FGD by products in highway applications will not be realized until the swelling nature of those materials is fully understood and methods developed to control swelling (Hopkins et al 1993a and Hopkins and Beckham 1995).

Another objective of past research (Hopkins et al 1995) involved developing methods for rapidly evaluating the in situ bearing strengths of untreated and treated subgrades. The dynamic cone penetrometer and the Clegg impact hammer were selected for evaluation. Many dynamic cone penetrometer tests, in situ CBR tests, and unconfined compression tests were performed on newly constructed highway subgrades. Correlations were developed between dynamic cone penetrometer values, unconfined compressive strength, and CBR tests. Additionally, Clegg impact hammer values were correlated with unconfined compressive strengths. These correlations have been used by engineers of the Kentucky Transportation Cabinet to obtain a rapid evaluation of the strength characteristics of treated and untreated highway subgrades.

Chemical admixture specifications include a stipulation that the temperature must be greater than 7.2°C (45° F) before chemical stabilization is allowed. When the air temperature is below about 4.4 to 7.2 degrees Centigrade (40 to 45 degrees Fahrenheit) at the time of chemical stabilization, chemical reactions between soil particles and hydrated lime or cement may not occur. Consequently, improvement in bearing strength of the treated subgrade will not occur and alternate stabilization methods may be required.

SURVEY OF SUBGRADE STABILIZATION PRACTICES IN THE UNITED STATES

To determine subgrade stabilization practices in the United States, a survey was sent to all states to determine if they used subgrade stabilization and, if so, what type of stabilization is used and what is the criteria for using stabilization. Thirty-eight states (including Kentucky) responded to the survey, as shown in Figure 1. Twelve states and the District of Columbia did not respond to the survey. Survey responses from each of those states are summarized in tables in APPENDIX A. All states that responded use mechanical stabilization and have soil compaction standards, which specify certain values of density and moisture that must be achieved. Typically, states require that 95 to 100 percent of maximum dry density obtained from standard moisture–density relations, similar to those obtained from AASHTO T-99, or 90 to 100 percent of maximum dry density obtained from modified moisture-density relations, similar to those obtained from AASHTO T-180. Some states accepted proof rolling and/or compaction. Several states indicated that soft soils are often stabilized mechanically by removing the soft, or unsuitable, soil and replacing the undesirable soil with crushed stone. In some instances, the stone is reinforced with either geogrids or geofabrics.

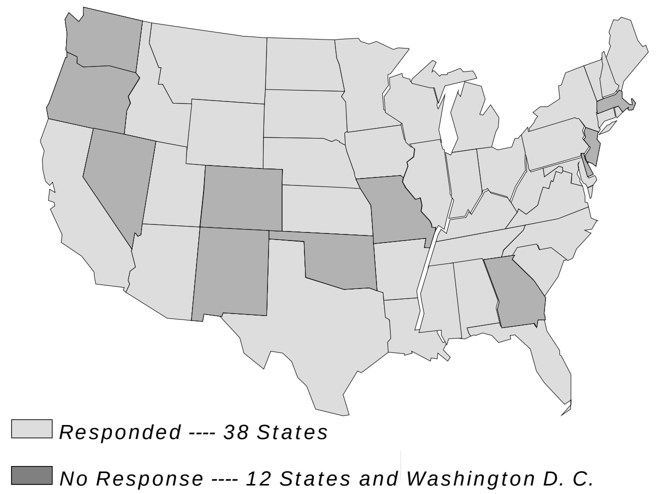

Figure 1. Survey response of states using chemical subgrade stabilization.

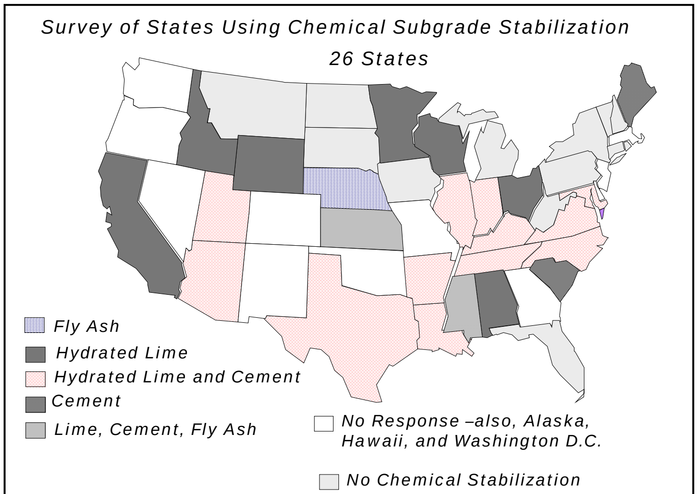

Figure 2. Chemical admixtures used by different states.

Twenty-six of the 38 states use chemical admixtures to improve the bearing strengths of soil subgrades. Those states use hydrated or quick lime, Portland cement, fly ash, or combinations of these agents for stabilization, as illustrated in Figure 2. Kentucky, Illinois and Indiana noted that lime kiln dust—a byproduct from the production of hydrated lime—had been used on occasions. Although the survey showed that chemical stabilization is used widely in the United States, 13 states indicated that they do not use chemical stabilization. However, 12 of the 13 states that do not use chemical stabilization are located in the northern part of the continental United States where low temperatures reduce the construction season. Since the temperature must be greater than approximately 45^0 F for chemical reactions to occur in soils and the low seasonal temperatures in the northern states, the opportunity to use chemical stabilization is shorter than in more southern states of the country. Florida noted that chemical stabilization is rarely used. This state stated that it had an abundance of lime rock, which makes it an economical stabilizer. Some states indicated that subgrade stabilization is not an issue in the state, or it is not used.

All of the states that responded to the survey indicated that chemical and mechanical stabilization was beneficial. The most cited benefit by the majority of the states was that stabilization "provides a uniform construction platform and foundation for pavement structure," as noted by Alabama. Kansas noted that stabilization "provides all weather working platform, increased performance life of pavement...cost effective for reducing paving materials, and promotes reconstruction." Maine noted that stabilization retards frost heaving while North Carolina stated, "chemical stabilization reduces moisture susceptibility problems." Michigan noted that a "stable subgrade is essential to maintaining integrity of base course." Texas provided the best answer to the benefit of stabilization when they stated " Yes....we believe in building pavements from bottom up and pay special attention to the subgrade as we will probably never see it again."

Twenty-eight of the 38 states give structural credit to both mechanical and chemical stabilization. For example, Alabama indicated that an AASHTO (1993) structural layer coefficient equal to 0.10 was assumed when hydrated lime stabilization is used. A value of 0.05 is assigned to select subgrade material. Arkansas increases the structural number, SN, 0.07 per inch of soil-hydrated lime stabilized depth and 0.20 per inch of soil-cement stabilized depth. Illinois gives structural credit when "stabilization' is used. When the soil is merely "modified,"—the use of a small percentage of admixture-- no structural credit is given. Illinois did not supply values of layer coefficients. Kansas indicated that a structural layer coefficient of 0.11 is used when lime stabilization is used. South Carolina used a structural layer coefficient of 0.15 for soils treated with Portland cement. California noted that the stabilized subgrade is "considered to have properties of an aggregate base." Arizona adds 10 points to subgrade R-value when stabilization, geogrids, or geofabrics are used. Florida assigns a value of 0.08 when the subgrade is stabilized. Although several states indicated that structural credit is given to the stabilized subgrade, they did not supply values of layer coefficients assigned in their pavement designs. Some states did not give structural credit because they did not stabilize the subgrade, or it was not an issue in their state.

Several states use chemical and mechanical subgrade stabilization for "poor" or "low- strength soils." California uses hydrated lime to treat fat clays when the R-value is less than 10. Quick lime was not used as frequently to treat those types of clays. Arizona indicated that chemical (hydrated lime and Portland cement) and mechanical stabilization (geogrids and geofrabric) were used when the R-value was less than 15. Some states used chemical stabilization when the soil subgrades were "wet" to expedite construction and prevent "delays due to wet subgrades," as noted by Kansas. Chemical admixtures were used to dry the soils and to provide a good working platform during construction.

SELECTION AND LOCATION OF STUDY SITES

Field Reconnaissance

Subgrade Stabilization Methods used in Kentucky

Many methods have been used to stabilize, or improve, the bearing capacity of subgrades. Basically these methods can be broadly divided into two groups: mechanical and chemical. Chemical admixtures used in Kentucky include Portland cement, hydrated lime, and such byproducts as lime kiln dust (LKD), and atmospheric fluidized bed combustion ash (AFBC). LKD is a byproduct obtained in the production of hydrated lime. The AFBC byproduct is produced by an oil refinery in Kentucky and also by coal-fired power plants. Typically, chemical admixtures used for subgrade stabilization in Kentucky are Portland cement and hydrated lime. Mechanical stabilization includes compaction, excavation of the top portion of subgrades and replacement with crushed stone, or crushed stone reinforced with geosynthetics. This study focused on the use of chemical admixtures for improving the bearing strength of soil subgrades. However, some attention is focused on the long-term behavior and performance of compacted (untreated) soil subgrades. Reinforced bases have been used at some sites. However, this stabilization technique is not included in this study. It has been described elsewhere (Hopkins and Beckham 1995, Hopkins and Slepak 2002).

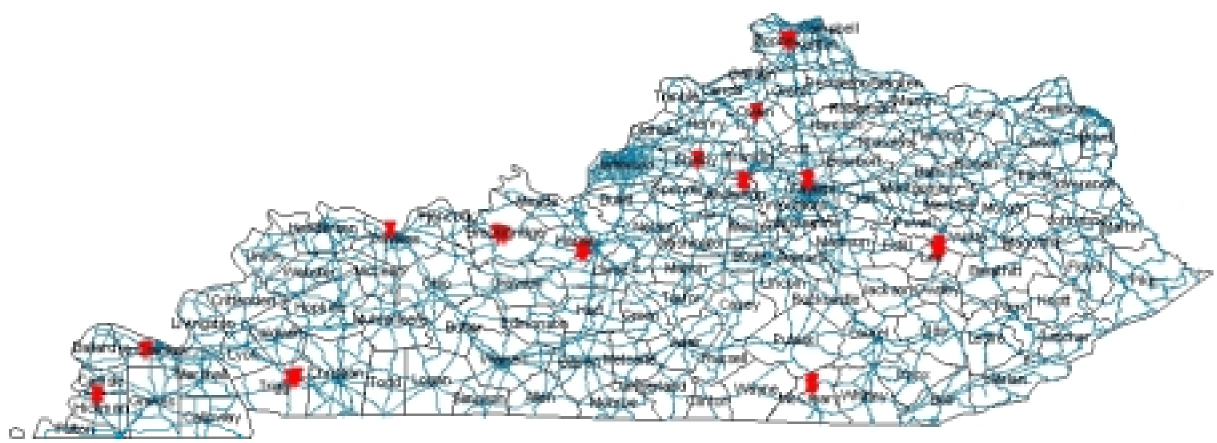

Figure 3. General locations of study sites in Kentucky.

Locations and Attributes of Stabilized Highway Subgrade Sections

Identifying and physically locating a statistically and significant number of highway pavement sections containing soil subgrades treated with chemical admixtures was a cooperative effort of personnel of the Kentucky Transportation Cabinet and the University of Kentucky Transportation Center. The Kentucky Transportation Cabinet provided a list of thirteen roadway sites where they wanted detailed investigations. The University of Kentucky Transportation Center provided another site containing seven additional sections where studies had been conducted for several years. Cabinet engineers have reportedly used chemical stabilization at more than 100 sites since 1987. All study sites were located according to milepost numbers. However, personnel of the Cabinet knew only approximate values of starting and ending mileposts of each section. General locations of the study sites are shown in Figure 3. Characteristics, including route numbers, lengths of each section, and an approximate date of construction and age are summarized in Table 1.

Coring Technique and Field Testing Procedures

Core holes were drilled approximately every tenth of a mile within each study section. Special coring techniques were developed to avoid using water. Compressed air, instead of water, was used to advance the drill down to the top of the subgrade of each section. By using compressed air as the drilling media, soaking and softening of the top of the subgrade at each hole was prevented. Hence, the subgrade as it exists in its natural setting was preserved and undisturbed. Typically, four holes were drilled at each location. The first core hole was drilled to measure the thicknesses of the asphalt, aggregate base, and stabilized subgrade layers of the flexible pavement section. After removing and measuring the thickness of the asphalt core, the base aggregate was removed by hand to expose the top of the stabilized subgrade (or in some cases

Table 1. Listing of Pavement Sections and Attributes.

County	Route Number	Section Length (Miles)	Chemical Admixture	Date Built
Anderson	US 127	2.3	Hydrated Lime	1991
Boyle	US 127	1.9	Hydrated Lime	1990
Fayette	US 25	1.6	Hydrated Lime	1994
Lee/Wolfe:	KY 11			1987
Section 1		1.1	Atmospheric Fluidized Bed Combustion Ash (AFBC[1])	
Section 2		0.6	Portland Cement	
Section 3		1.0	Hydrated Lime	
Section 4		0.5	Lime Kiln Dust[2]	
Section 5		1.8	Portland Cement	
Section 6		0.2	Untreated	
Section 7		0.8	AFBC[1]	
McCreary	US 27	2.0	Portland Cement	1989
Shelby	KY 55	1.4	Hydrated Lime	1991
Hardin	US 62	3.1	Hydrated Lime	1989/1992
Owen	US 127	1.2	Hydrated Lime	1991
Trigg	US 68	3.5	Hydrated Lime	1994
Boone	KY 842 (US 25-42 Connector)	2.4	Hydrated Lime-Cement	1987/ 1988
McCracken	US 62	1.3	Lime Kiln Dust[2]	1990
Hickman	US 51	1.3	Lime Kiln Dust[2]	1990
Breckinridge	US 60	2.3	Portland Cement	1987
Daviess	KY 331 (River Port Access Road)	0.3	Portland Cement	1986

1. A byproduct produced by an oil refinery in Kentucky.
2. A byproduct resulting from the production of hydrated lime.

the top of the untreated subgrade). The depth, or thickness, of the aggregate base was noted. Then a standard penetration test (SPT) was performed on the stabilized subgrade to obtain a split spoon specimen of the stabilized subgrade. Phenolphthalein was applied along the length of the split spoon specimen to determine the portion of the specimen that had been stabilized. The stabilized portion of the core turns to a reddish color when phenolphthalein is applied. Thickness of the stabilized subgrade was noted.

At the same location, a second hole was drilled. After augering through the flexible pavement and aggregate base and exposing the top of the stabilized subgrade, an in situ CBR test was performed, as shown in Figure 4. After completing the CBR test, a moisture content specimen was obtained at the top of the stabilized subgrade. Augering continued down through the stabilized subgrade to the top of the untreated subgrade below the stabilized layer. A second in situ CBR test was performed on the untreated subgrade and a moisture content was obtained at the top of the untreated subgrade. The SPT and in situ tests were performed according to test designations listed in Table 2. A third hole was advanced through the asphalt layer and aggregate base and a thin-walled, undisturbed sample, or a core specimen was obtained of the stabilized subgrade. Thin-walled tube samples of the stabilized subgrades could not be obtained in many cases. In this case, core specimens were obtained. A fourth hole was augered down through the asphalt layer, the aggregate layer, and stabilized layers to exposed the untreated layer below the stabilized layer. A thin-walled tube sample was obtained of the nonstabilized subgrade. Latitudes and longitudes of each section and borings within each section were determined using mapping-grade, GPS (Global Positioning System) equipment. Accuracy of the locations of holes was within a sub meter of the true location. The latitude and longitude of each core hole of each section are summarized in Appendix B. During the fieldwork, some 355 borings were made in the study sections. This number does not include the numerous borings performed over a period of several years in the study sections of KY 11 in Lee-Wolfe Counties. Falling weight deflectometer (FWD) tests were performed on each study section.

Figure 4. Performing in situ CBR tests.

TESTING METHODS

A variety of laboratory tests were performed, as summarized in Table 2. All tests were performed in accordance with AASHTO standards and designations. Laboratory tests included moisture content, liquid limit, plastic limit, specific gravity, grain-size analysis, Unified and AASHTO soil classifications, unconfined triaxial compression, and resilient modulus. Generally, the index tests (liquid limit, plastic limit, specific gravity, and grain-size analysis) were performed on the split-spoon samples and thin-walled specimens. Resilient modulus tests were performed on the core specimens and undisturbed thin-walled tube samples of the untreated subgrades. Unconfined triaxial compression tests were usually performed on the specimens after completion of the resilient modulus tests. Although resilient modulus tests were performed on the thin-walled tube specimens obtained from the stabilized subgrade, the results of those tests were not included in this

Table 2. Listing of field and laboratory tests.

Type of Test	**Test Designation**
In Situ CBR	ASTM D 4429
Standard Penetration Test (SPT)	AASHTO T 206
Moisture Content	AASHTO T 265
Liquid Limit	AASHTO T 89
Plastic Limit	AASHTO T 90
Specific Gravity	AASHTO T 100
Grain-Size Analysis	AASHTO T 88
Unconfined Triaxial Compression Test	AASHTO T 208
Resilient Modulus	AASHTO T 292 & AASHTO T 307

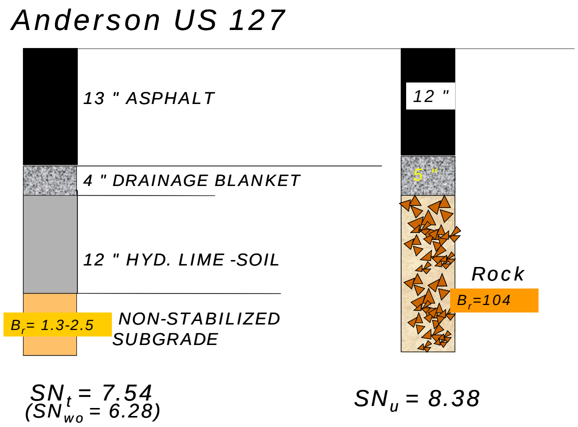

Figure 5. Pavement sections at the Anderson County site, US 127.

report. Test specimens of stabilized subgrades obtained from tube samples were of very poor quality. These specimens were very brittle and fractured. Specimens obtained by coring the stabilized subgrade using compressed air generally produced high-quality test specimens for resilient modulus testing. Those data were included in this report.

SUBGRADE ATTRIBUTES AND FIELD TEST RESULTS

Anderson County, US 127

The selected section of roadway for analysis begins just north of the intersection of Route US 127 with KY 151 at Mile Post (MP) 8.897 and ends at the Anderson-Franklin County line at MP 11.120.

Table 3. US 127 in Anderson County

MP	AC Thickness (in.)	Crushed Stone Drainage Base Thickness (in.)	Lime Stabilized Subgrade Thickness (in.)	Stabilized CBR	Non Stabilized CBR	SPT Blows per 6-in. intervals
9.1	13.0	4.0	12.0	19.0	1.3	7 / 4/ 5
9.8	13.5	4.0	12.0	55.0	2.5	8 / 4/ 4
10.1	13.0	4.0	24.0	39.5	NA	6 /10/ 8
10.4	13.0	5.0	12.0	60.0	1.52	4 / 3/ 2
10.7	12.0	5.0	Rock Subgrade	104.0		25/22/26

This section of roadway is a four-lane divided highway. Prior to construction, this section of roadway, which was completed in 1991, was a two-lane undivided highway. The roadway was reconstructed and two lanes were added. The new lanes are now the southbound lanes, and the existing lanes were converted to carry northbound traffic. Only the southbound lanes were evaluated. The subgrade on the northbound lanes was not stabilized.

A lime stabilized subgrade, which measured 12 inches in thickness, was constructed from MP 8.897 to about MP 10.53, except in some cut areas where excessive moisture was encountered. A typical section is shown in Figure 5. Those areas

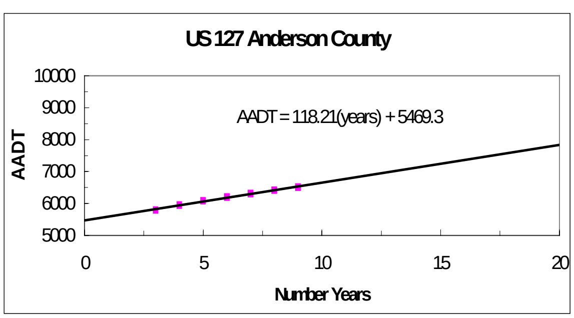

Figure 6. Relationship of AADT and time.

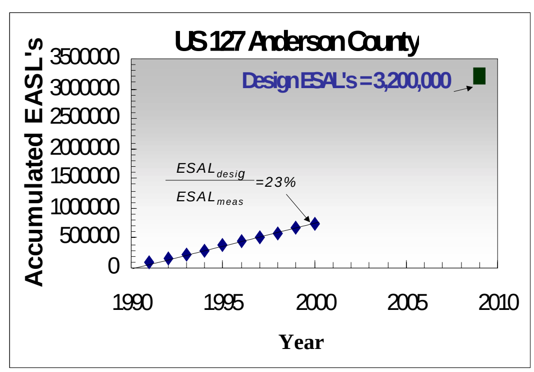

Figure 7. Accumulated ESALs as a function of time.

were stabilized to depths ranging from 24 to 48 inches. The lime stabilized subgrade was originally designed to be eight inches thick using four percent (dry mass) hydrated lime from Stations 501 + 76 to 531 +50 and five percent from Station 531 +50 to 579 +00. A crushed rock roadbed, as shown in Figure 5, was used on the northern end of the new section from approximate MP 10.53 to the Anderson-Franklin County line (MP 11.120). Exact location of the rock subgrade and lime-stabilized soil subgrade interface could not be determined. The section had a design life of 20 years and 3,200,000 ESALs (Equivalent Single-Axle Loads).

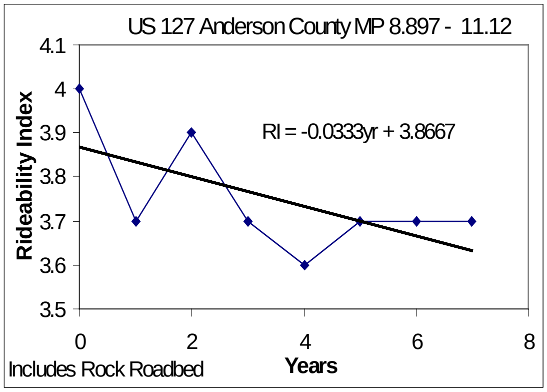

Figure 8. Rideability index as a function of time.

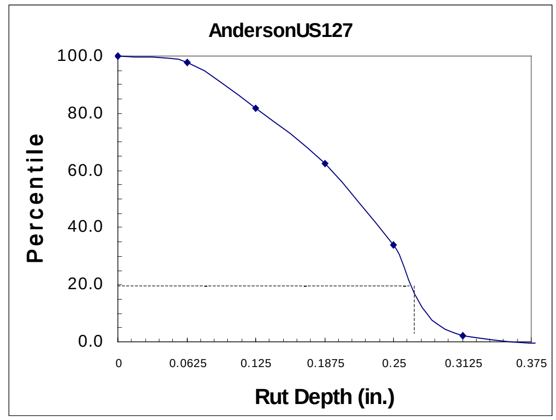

Figure 9. Percentile test value as a function of pavement rutting depth (in.)

Pavement thickness, in situ California Bearing Ratio (CBR), and Standard Penetration Test (SPT) values measured during the field study are summarized in Table 3. Standard Penetration Tests were performed on the top of the stabilized subgrade. The values shown are the number of blows per 6-inch increments. Samples recovered from SPT were used to determine the thickness of the stabilized subgrade.

The trend of the relationship of average annual daily traffic (AADT) and time is shown in Figure 6. AADT is increasing with increasing time. Values of ESAL were determined using a program developed by Rister and Allen (1999). ESALs as a function of time are shown in Figure 7. Approximately 23 percent of the design life of 3.2 million ESALs have been used to date at this site. Rideability index (RI) as a function of time for this section is shown in Figure 8. The initial value of RI was 3.86 and decreases with increasing time. The current value of RI is about 3.63 and projected values in 15 and 20 years are 3.37 and 3.2, respectively. Rutting measurements of this section of roadway are shown in Figure 9 in the form of percentile test value as a function of rut depth. At the 20th percentile test value the depth of rutting of the section is about 0.27 inches.

Boyle County, US 127 Bypass (NBL Only)

The section selected for evaluation begins at the intersection of US route 150 and extends to the intersection of US 127 (MP 3.196 – 5.270. The road is a divided four-lane highway. The

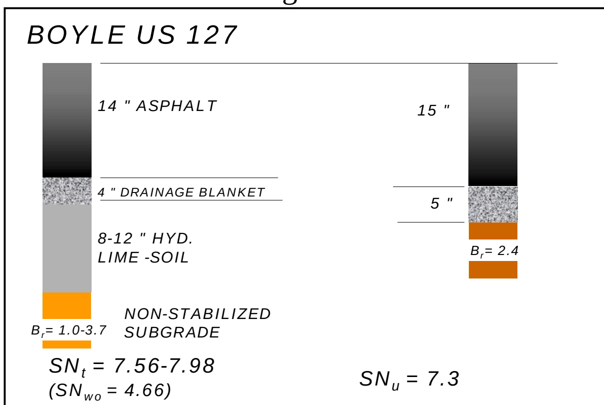

Figure 10. Pavement section at the Boyle County site, US 127 By Pass.

northbound lanes were tested. This section was constructed in 1990. The subgrade was stabilized with five percent (by dry mass) of hydrated lime. The design depth was eight inches. However, the measured thickness of the stabilized subgrade ranged from 8 to 12 inches. One of the test locations, MP 5.1, was not stabilized and was apparently beyond the limits of the section constructed with a stabilized subgrade. A cross section of the non-stabilized section is shown in Figure 10. The section had a design life of 20 years and 9,200,000 ESALs.

Section thickness, CBR data of the stabilized and non-stabilized subgrade, and SPT values are summarized in Table 4. Relationship between values of AADT and time and estimated accumulated ESALs and time are shown in Figures 11 and 12, respectively. In both cases, the AADT and

Table 4. Boyle County, US 127 By Pass

MP	AC Thickness (in.)	Crushed Stone Drainage Base Thickness (in.)	Lime Stabilized Subgrade Thickness (in.)	Stabilized CBR	Non Stabilized CBR	SPT Blows per 6-in. intervals
3.4	14.0	4.0	12.0	40.8	2.4	8/10/13
3.65	14.0	4.0	12.0	29.3	3.7	6 / 6/ 6
4.1	15.0	4.0	8.0	16.5	1.0	6 / 4/12
4.3	14.0	5.0	8.0	64.3	2.1	7 / 5/ 6
4.6	14.0	4.0	8.0	50.0	2.7	5/ 5/ 10
5.1	14.0	5.0	None		2.4	7/ 7/ 8

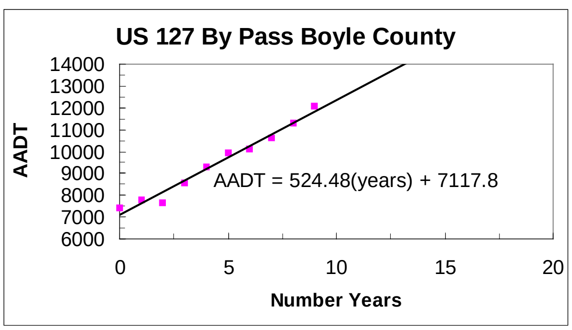

Figure 11. AADT as function of time.

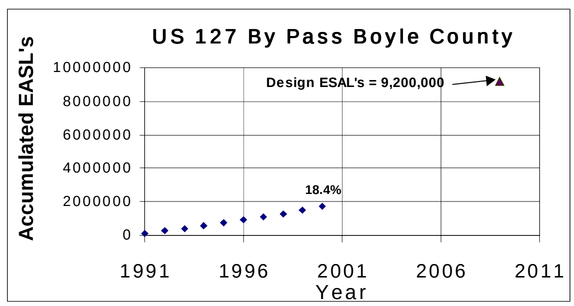

Figure 12. ESALs as a function of time.

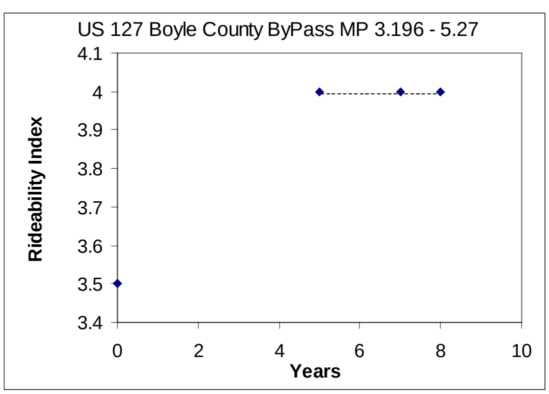

Figure 13. Rideablity indices.

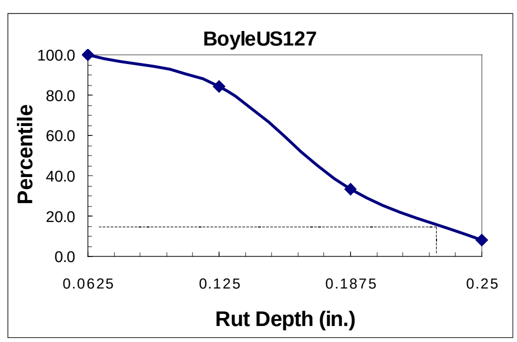

Figure 14. Average rut depth as a function percentile test value.

accumulated ESALs are increasing with increasing time. About 18 % percent of the design life (9.2 million ESALs) have been used at this site. Estimated AADT at the end of the 20-year design period is about 17,600 cars per day. Only three values of rideability index have been recorded at this site, as shown in Figure 13. All of those values are 4. As shown in Figure 14, the average rut depth at the 20th percentile test value is about 0.23 inches, or 80 percent of the section had rutting depths less than 0.23 inches.

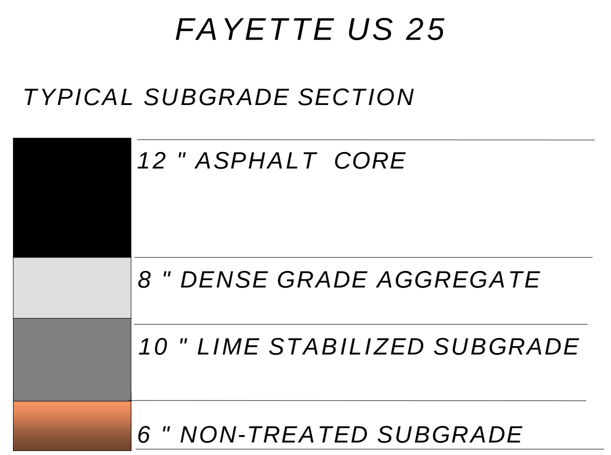

Figure 15. Pavement cross section at the Fayette County site, US 25.

Fayette County, US 25

This section of US 25 was reconstructed in 1994. It begins at the intersection of KY 4 (New Circle Road) and US 25 at MP 16.236. The section ends at the I-75 Overpass (MP 19.031). A typical section is shown in Figure 15. During the field investigation, several pavement sections were discovered. Portland cement concrete (PCC) pavement was constructed from MP 16.236 to 16.76. Results of field tests at one location near MP 16.7 (SBL) within the PCC pavement section showed that the pavement section consisted of 16.7 inches of treated PCC and 5 inches of an asphalt-treated drainage layer resting on geogrids. CBR of the untreated subgrade was 7.5. Other test locations revealed different pavement thickness and either chemical stabilization using hydrated lime (5%) or mechanical stabilization using geogrids. The test location at MP 18.3 was not in the reconstructed section.

Table 5. Results of field tests at the US 25 site in Fayette County.

MP	A. C. Thickness (in.)	Stone Base Thickness (in.)	Stabilization Type	Stabilized CBR	Non Stabilized CBR	SPT Blows per 6-in. intervals
16.95 NB	19.0	5.0	Geogrid		7.4	12/ 3/ 6
17.14 SB	12.5	5.5	8 in. Lime	8.8	6.9	2 / 3/ 7
17.35 NB	10.5	8.0	Geogrid		8.1	5 / 5/ 6
17.50 SB	12.5	5.0	8 in. Lime	26.0	1.2	8/ 7/ 8
17.70 NB	10.0	8.0	None		5.2	4/ 6/ 18
17.90 SB	10.0	8.0	8 in. Lime	12.3	4.5	3/ 4/ 7
18.10 NB	9.5	8.5	8 in. Lime	32.3	2.5	4/ 4/ 5
18.25 SB	10.0	8.0	10 in. Lime	32.5	3.0	9/ 9/ 6
18.30 NB	12.0	8.0	10 in. Lime	77.5	0.5	12/ 9/ 5
18.90 NB	12.0	8.0	None		NA	NA

The recommendation to use five percent hydrated lime was based on past experience. Most clay soil evaluated over the last few years required five percent hydrated lime to achieve the desired increase in strength. The amount of additive to use is based on the increase in unconfined compressive strength. No testing was performed to determine the percent of lime to use on this project. A soil sample was obtained from this project during construction. Unconfined compressive

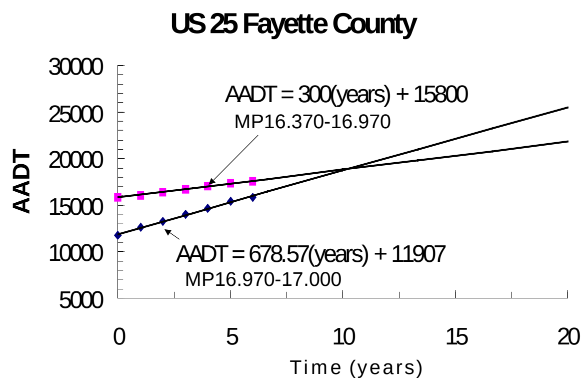

Figure 16. AADT as a function of time.

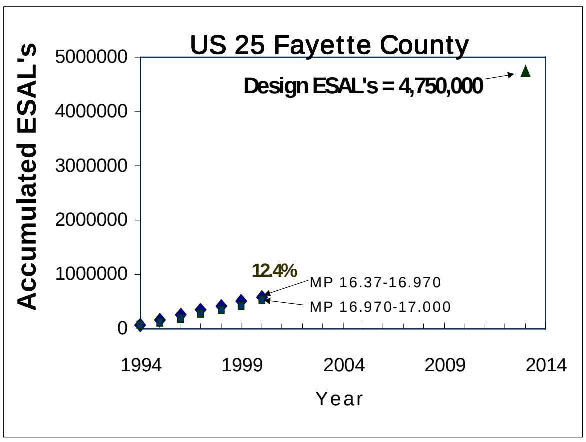

Figure 17. Accumulated ESALs as a function of year.

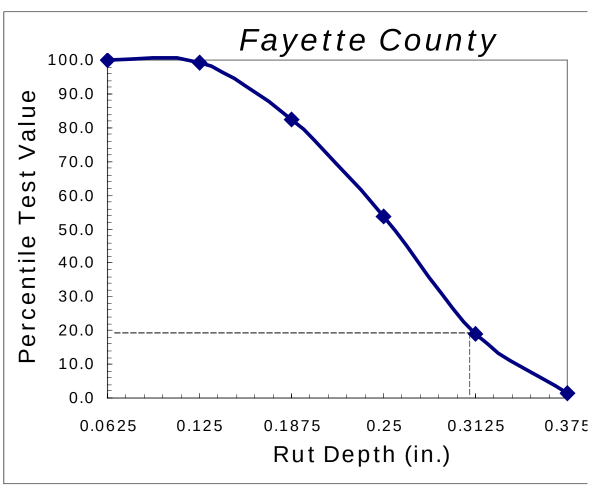

Figure 18. Average rut depth (inches).

strength tests indicated that unconfined compressive strength increased and was greater than the strength observed at five percent of hydrated lime. Various thickness of pavement layers at different hole locations, stabilizing methods observed from the field study, and CBR values of the stabilized and non-stabilized subgrades are summarized in Table 5.

Values of AADT and accumulated ESALs as a function of time are shown in Figures 16 and 17, respectively. Projected values of AADT for a twenty-year period range from about 22,000 to 25,500. The design value of ESAL for this pavement was 4.75 million. About 13 percent of the design ESAL has occurred at this site. The average rideability index of the section, as measured in 1999, was 3.6. At the 20th percentile test value, the depth of rutting is about 0.3 inches, as shown in Figure 18.

Lee-Wolfe Counties, KY 11

This section of roadway was reconstructed in 1987. Initially, it was established as an experimental research study to examine the long-term durability of stabilizing soil subgrades with chemical admixtures (Hopkins et al 1988 and Hopkins et al 1995). The reconstruction project began at the intersection of KY 11 and KY 498 (MP 9.423) in Lee county and ends at the intersection of KY 11 and KY 715 in Wolfe County. Actual station numbers were 260+00 to 422+00 and 422+00 to 576+60. The soil subgrade in the 6-mile long roadway was to be initially designed as a working platform to facilitate construction. It was to be stabilized with ten percent (by dry mass) Portland cement. Before stabilization began, a change order was issued that allowed the use of a lime by-product material (Atmospheric Fluidized Bed Combustion--AFBC) as a substitute for Portland cement. A decision was also made to use other types of chemical subgrade stabilizing materials, such as hydrated lime, Portland cement, and lime kiln dust, a byproduct produced from the manufacturing of hydrated lime.

Table 6. KY 11 subgrade experimental stabilization sections.

Subgrade Chemical Admixture	Length (Miles)
AFBC Ash	1.1
Portland Cement	0.6
Hyd. Lime	1.0
Lime Kiln Dust	0.5
Portland Cement	1.8
Untreated	0.2
AFBC	0.8

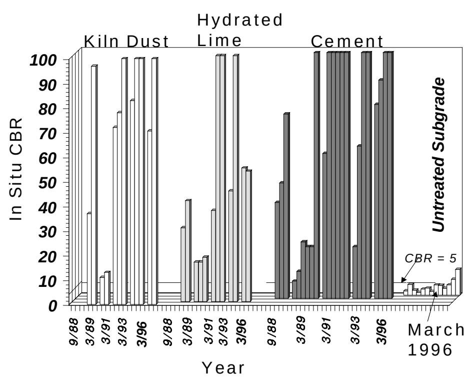

Figure 19. In situ CBR values of stabilized kiln dust, hydrated lime, cement and non-stabilized subgrades of the KY 11 sections.

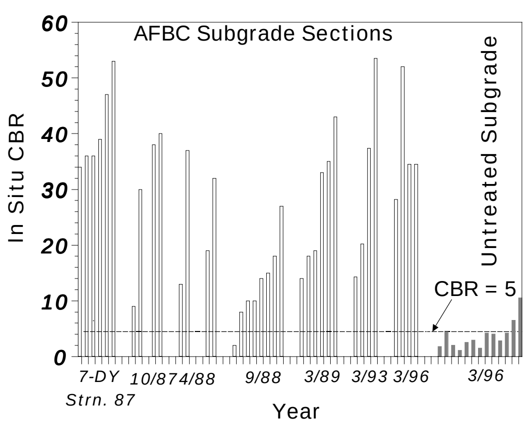

Figure 20. In situ CBR values for the AFBC byproduct subgrade sections.

The 6.0-mile reconstruction project was divided into seven sections. Six subgrade sections were stabilized with different chemical admixtures and one subgrade section was left untreated. The type of chemical admixture and length of each section are summarized in Table 6. The main intent of the experimental research study was to examine the long-term durability of chemical stabilization. This site has been monitored over the last 15 years (monitoring started during construction in 1986 and has continued to the present time, 2002). Field studies have been conducted to observe the change in the in situ CBR and moisture content of the stabilized and non-stabilized subgrades with increasing time. Values of in situ CBR measured during the period 1987-1996 for subgrades treated with lime kiln dust, hydrated lime, and cement are compared to in situ CBR values of the untreated subgrade (located below the treated subgrade) in Figure 19. The in situ CBR value is typically below 5. In situ CBR values of the treated subgrades are several times larger than the untreated subgrades. In situ CBR values of the subgrades mixed with the AFBC byproduct for the period 1987-1996 are shown in Figure 20. Generally, the CBR values of the AFBC sections range from 12 to 51. CBR values of the other stabilized subgrades ranged from 94 to a value in excess of 100.

Initially, moisture contents of the untreated subgrade occurring at the 85th and 50th percentile test values, as shown in Figure 21, were only about 6.9 and 12.0 percent, respectively. About two years later (1989), the moisture contents at the same percentile test values had increased to 12.9 and 17.3 percent. By 1991, the values had increased slightly to 14.5 and 18.0 percent, respectively. However, by 1993, the moisture contents at the same percentile test values increased significantly to 16.3 and 20.0, respectively. Moisture contents of the top of the untreated subgrade located below the stabilized subgrades are some 1.5 to 2.2 times greater by 1993 than the moisture contents at the time of construction.

Field data obtained during the recent study are summarized in Table 7 and include milepost location, thickness of the asphalt pavement, aggregate base, and stabilized layer, CBR values of

Table 7. Listing of boring data obtained in the summer of 2000 at the Ky 11 site in Lee and Wolfe Counties.

MP	A. C. Thickness (in.)	Stone Base Thickness (in.)	Stabilization Type	Stabilized CBR	Non Stabilized CBR	SPT Blows per 6-in. intervals
10.0	9.0	5.0	12 in. AFBC	51.3	3.2	10/ 9/ 4
10.2	9.0	5.0	12 in. AFBC	26.0	4.1	6/ 4/ 5
10.4	7.0	5.0	12 in. cement, 10%	137.5	3.9	20/24/ 6
10.5	7.0	5.0	12 in. cement, 10%	98.5	1.6	16/ 9/ 5
11.0	10.0	8.0	None		5.2	4/ 6/ 18
11.2	9.0	5.0	12 in. Lime	93.5	1.4	24/41/60
12.0	9.0	4.0	12 in. lime kiln dust	122.7	3.7	14/10/ 5
12.5	9.0	4.0	11 in. lime kiln dust	104.5	2.2	22/14/ 7
13.7	7.5	5.0	12 in. cement, 7%	106.0	2.7	31/31/17
14.1	10.0	6.0	None		6.8	3/ 4/ 7
14.5	7.5	5.0	12 in. AFBC	27.8	2.2	6 / 5/ 3
14.7	7.5	6.0	12 in AFBC	35.5	7.1	11/ 8/ 9

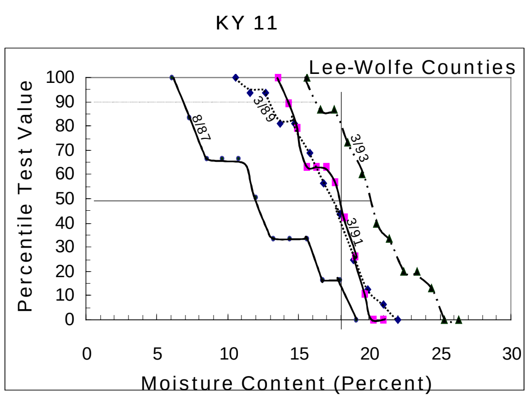

Figure 21. Percentile test value versus the moisture content of the top of the untreated subgrade.

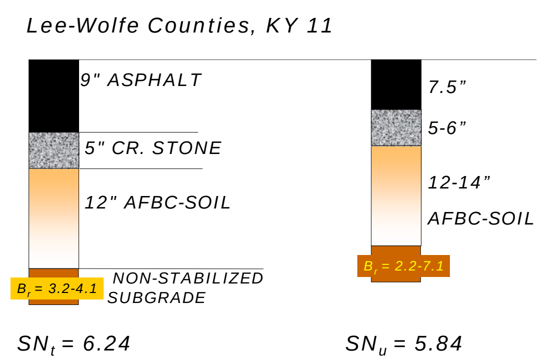

Figure 22. AFBC cross sections.

the stabilized and non-stabilized subgrades, and values of standard penetration tests. Typical cross sections of the flexible pavement of this 6-mile roadway section are shown in Figures 22 through 25. Values of CBR of the stabilized subgrades ranged from 26 to values in excess of 100. Values of CBR of the untreated subgrade were very low and ranged from 1.4 to 7.1. Average CBR values of the untreated and stabilized subgrades were 3.7 and 80, respectively.

The ESAL design value of this section was 1.3 million. The relationship between AADT and time is shown in Figure 26. The initial AADT was about 2000 and increases to about 2600 in 12 years. A projected value of AADT in twenty years is about 2934. Estimated accumulated values of ESAL, based on the AADT values, are shown in Figure 27. About 52 percent of the design value of ESAL has occurred at this site in 12 years.

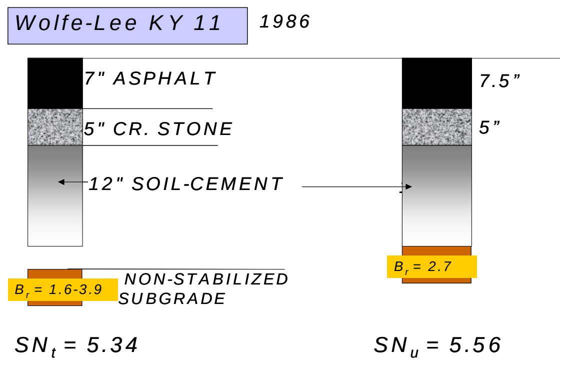

Figure 23. Soil-cement cross-sections.

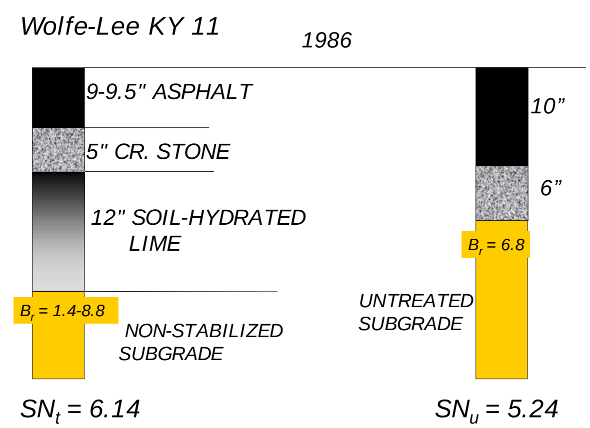

Figure 24. Soil-hydrated lime-soil and untreated cross sections.

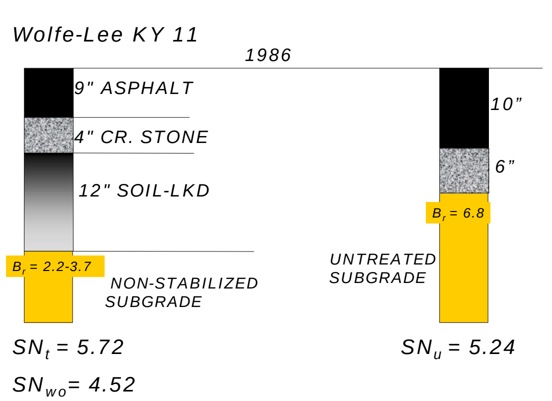

Figure 25. Kiln dust-soil cross sections.

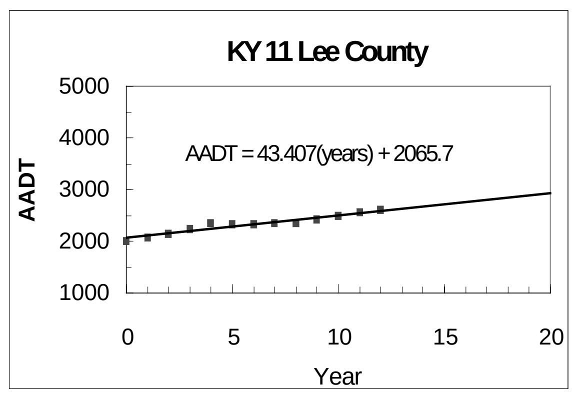

Figure 26. AADT as function of time, KY 11 in Lee-Wolfe Counties.

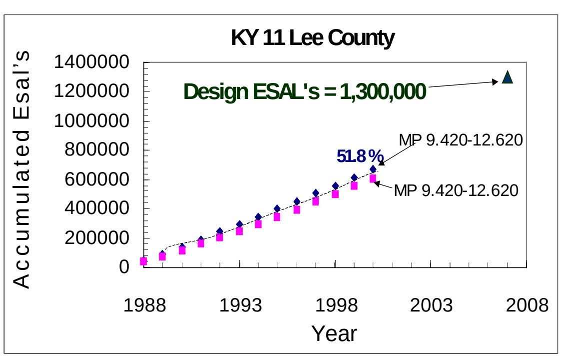

Figure 27. Accumulated values of ESALs as function of time, KY 11 in Lee-Wolfe Counties.

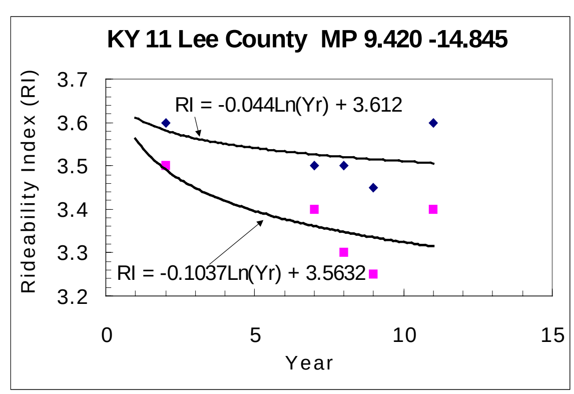

Figure 28. Rideability Index as a function of time for KY 11 in Lee-Wolfe Counties.

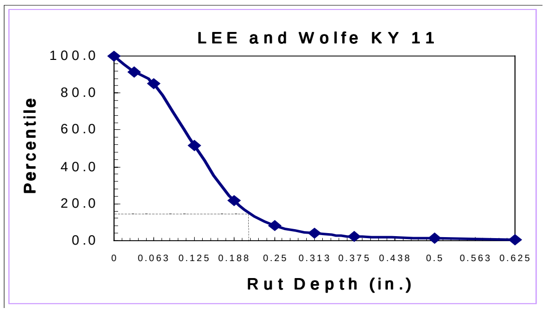

Figure 29. Relation between percentile test value and rut depth.

Rideability index as a function of time is shown in Figure 28. Initially, the RI values were about 3.5-3.6. After 12 years, and based on the trend relationships, the RI-values range from 3.31 to 3.50. Projected RI-values at 20 years range from 3.25 to 3.48. Rutting measurements are shown in Figure 29 as a function of percentile test value. At the 20th percentile test value, the average rutting depth is less than 0.20 inches.

McCreary County, US 27

This roadway section extends from Station 599+00 to 796+00. However, this stretch of roadway contains two sections. One portion begins at the intersection of US 27 and the Robert Bryant Road, MP 14.2, and ends at MP 18.159. The Division of Materials, Geotechnical Branch, recommended using six percent of Portland cement to stabilize the subgrade from stations 707+00 to 737+00. The second portion extends from Stations 742+00 to 787+00 and the Geotechnical Branch recommended that four percent of Portland cement be mixed with the subgrade of this section that was reconstructed in 1989. A typical cross section of the pavement is shown in Figure 30. At one location, the pavement was located on a rock subgrade. CBR of the rock subgrade was about 44.

Field data obtained during the study are summarized in Table 8. The data include milepost location, thickness of the asphalt pavement, aggregate base, and stabilized layer, CBR values of the stabilized and non-stabilized subgrades, and values of standard penetration tests. CBR values of the stabilized subgrades ranged from 37 to a value in excess of 100. Values of CBR of the untreated subgrade ranged from 4.4 to 7.9. The average value of the untreated subgrade was 5.7. The average CBR value of the stabilized subgrades was 75.

The section had a design life of 20 years and 3.3 million ESALs. The relation between AADT and time is given in Figure 31 and it is very linear. A projected AADT for a 20-year period is about 7,373. Initially, the AADT was about 5000. Based on the AADT values, an estimated relation between accumulated ESALs and time is given in Figure 32. After about eleven years, some 38 to 46 percent of the design life of this pavement has been used. Rideability index of the pavement at this section has remained large after ten years, as shown in Figure 33. After ten years, the value of RI is 3.60. Projected values, based on the linear relationship in Figure 33 and estimated for 15 and 20 years, are 3.52 and 3.43, respectively. The depth of rutting, as related to the percentile test value, is shown in Figure 34. The depth of rutting at the 20th percentile test value is about 0.21 inches.

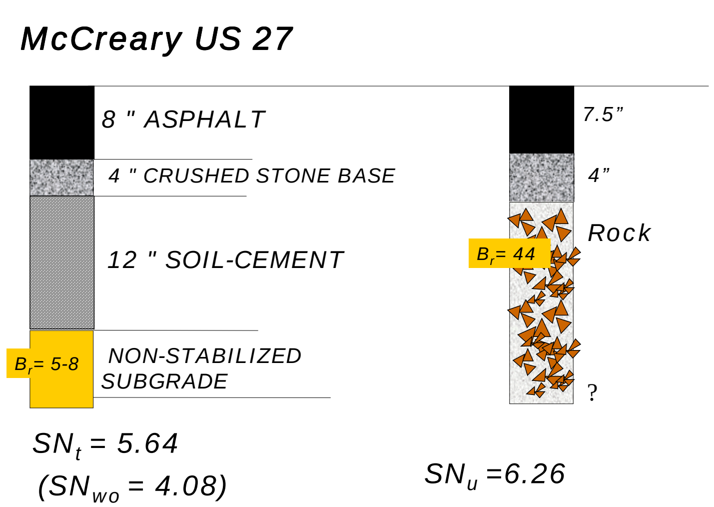

Figure 30. Cross sections, US 27, McCreary County.

Table 8. Listing of boring data obtained at the US 27 site in McCreary County.

MP	A. C. Thickness (in.)	Bank Gravel Base Thickness (in.)	Cement Stabilized Subgrade Thickness (in.)	Stabilized CBR	Non Stabilized CBR	SPT Blows per 6-in. intervals
15.23 NB	7.0	5.0	12.0	135.0	7.9	22/16/50
15.75 SB	7.5	4.5	8.0	73.5	5.8	23/11/12
16.75 NB	7.5	5.0	None		44.0*	17/25/21
17.20 NB	7.5	5.0	11.0	59.4	4.4	14/ 7/ 6
17.55 NB	8.0	5.0	10.0	37.0	5.0	22/11/10
17.80 SB	7.0	5.0	10.0	69.8	5.6	36/10/19

* Rock

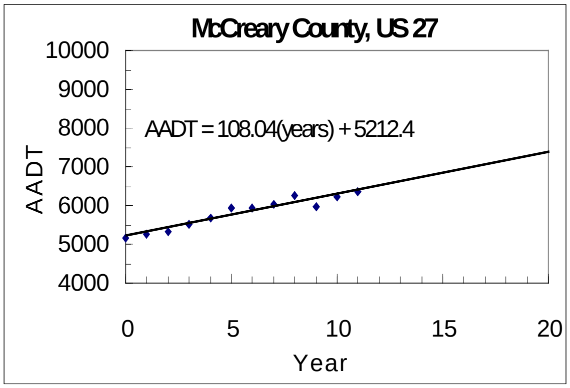

Figure 31. AADT as a function of year.

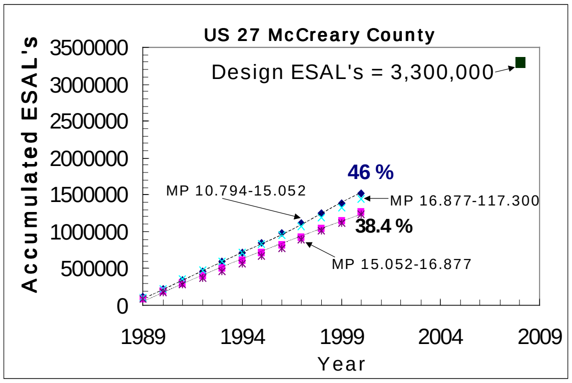

Figure 32. Accumulated ESALs as a function of time.

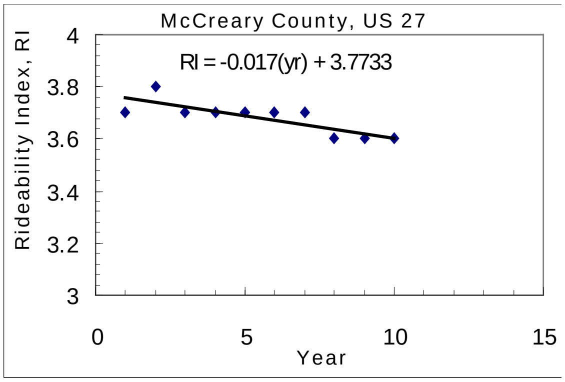

Figure 33. RI as function of time.

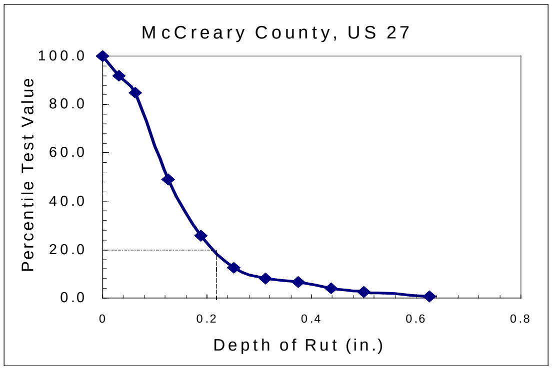

Figure 34. Percentile test value as function of depth of rutting (in.)

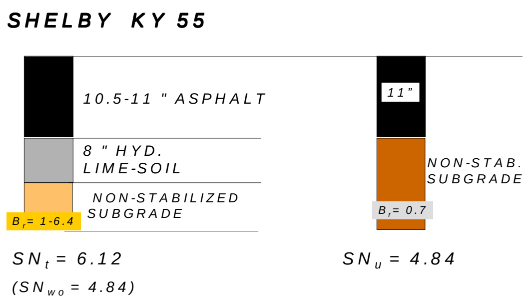

Figure 35. Pavement cross sections.

Shelby County, KY 55

Evaluation of this section began at the intersection of KY 55 and US 60, MP 7.898, and ends at MP 9.131, the intersection with KY 43 and 2268. Approximately 0.2 mile of the subgrade from MP 8.931 to 9.131 was not stabilized. This section was situated north of a railroad overpass. The project was designed to use Full Depth7 asphalt concrete. The pavement structure was designed as 11 inches of asphalt resting on 8 inches of a soil subgrade stabilized with five percent of hydrated lime. Project stations, as listed by record plans of the Kentucky Transportation Cabinet, were 123 + 00 to 153 + 00.

Thickness of the asphalt layer at this site ranged from 10.5 to 11.5 inches. Depth of the stabilized hydrated lime-soil layer was 8 inches. A drainage, or base aggregate, layer was not used at this site. Other drilling results are listed in Table 9. In situ CBR values of the stabilized subgrade ranged from 16.5 to 52.0 and averaged 26. The average value of CBR for the non-stabilized subgrade was 3.5. One boring occurred outside the stabilized areas. Cross sections of the borings in the stabilized and non-stabilized subgrades are shown in Figure 35.

Table 9. Drilling results for Shelby County, KY 55

MP	A. C. Thickness (in.)	Lime Stabilized Subgrade Thickness (in.)	Stabilized CBR	Non Stabilized CBR	SPT Blows per 6-in. intervals
8.15 NB	11.5	8.0	52.0	6.4	10/ 7/ 12
8.30 NB	11.3	8.0	18.8	4.2	4/ 4/ 9
8.50 NB	11.0	8.0	17.5	2.7	5/ 2/ 5
8.60 SB	10.5	8.0	16.5	1.0	3/ 6/ 9
8.85 SB	10.5	8.0	24.5	5.8	4/ 8/ 15
9.00 SB	11.0	8.0	None	0.7	2/ 6/ 4

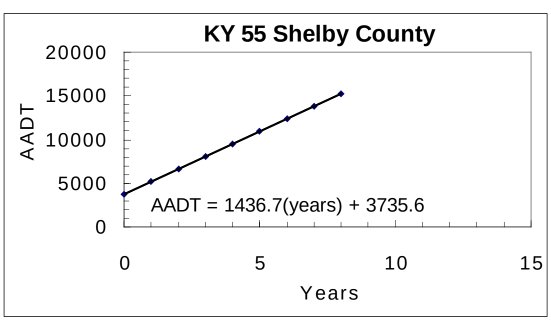

Figure 36. AADT as a function of time.

The section had a design life of 20 years and 2.4 million ESALs. The design CBR was 2. AADT as a function of time is shown in Figure 36. Although the beginning AADT-value was 3,736, the value has increased to 15,230 after 8 years. Projected values of AADT at the end of 15 and 20 years are 25,287 and 32,470, respectively. About 30 percent of the estimated, accumulated design ESALs have occurred at this site after 8 years, as shown in Figure 37. However, the growth of ESAL-values is exponential and it is estimated that the estimated ESAL values will exceed the design value after 20 years.

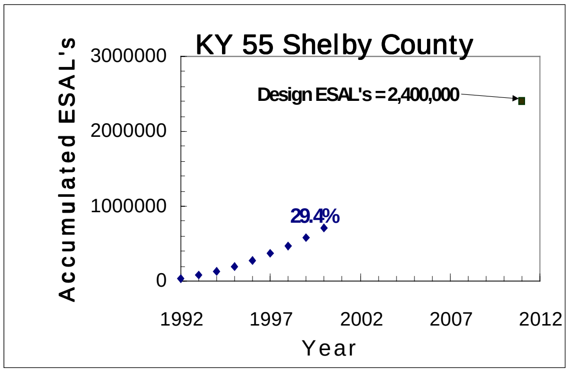

Figure 37. Accumulated ESALs as a function of time.

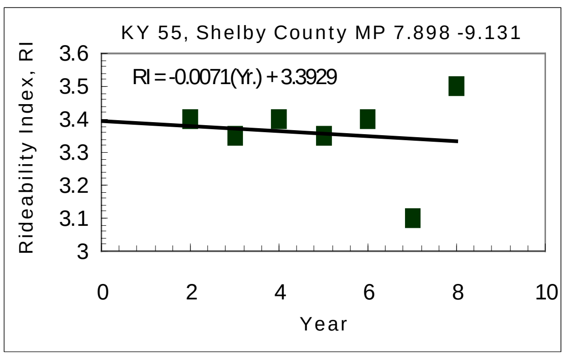

Figure 38. Rideability Index as a function of time.

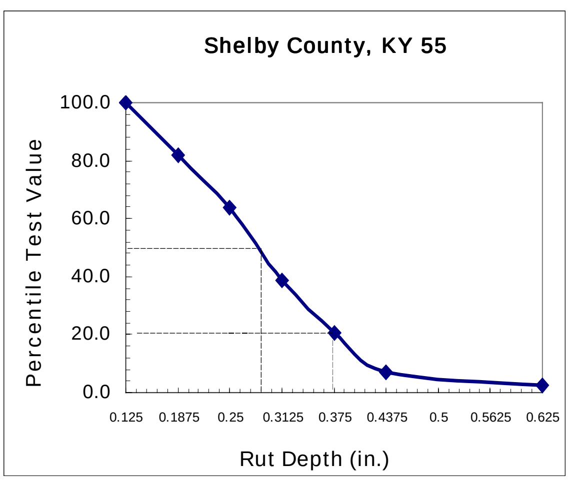

Figure 39. Percentile test value as function of depth of rutting.

Values of RI as a function of time are shown in Figure 38. The first value of RI recorded at this site occurred some 2 years after completion of construction and was only 3.4. After 8 years, the RI-value obtained from the trend relationship in Figure 38 is estimated to be 3.33. Projected RI-values after 15 and 20 years are 3.29 and 3.25, respectively.

Rutting measurements at this site are related to percentile test value in Figure 39. At the 50th and 20th percentile test values, the depths of rutting are about 0.29 and 0.38 inches, respectively.

Hardin County, US 62

This roadway is a four-lane divided highway. Two sections of roadway were evaluated. The westbound lanes of one section extending from station 150 + 00 to 185 + 00 were constructed in 1998. The eastbound lanes were constructed in 1989. The section was designed to have a 6-inch deep hydrated lime (six percent by dry mass) stabilized subgrade. During construction several wet areas were stabilized to depths of 16 inches or greater. Tests performed at MP 14.5 and 14.6 were, apparently, beyond the stabilized section. An additional section adjacent to the previous section was also tested. This section is one of three subgrade chemical stabilization sites statewide that are being evaluated periodically by the Kentucky Transportation Center as part of a long-term monitoring study of stabilized subgrades. The subgrade of this section was constructed in 1991. The KYTC project stations were 576 + 00 to 606 + 00 and 30 + 00 to 144 + 00, respectively.

Typical cross sections of the two different sections are displayed in Figure 40. Drilling results obtained from the two different sections of roadway are summarized in Tables 10 and 11. The asphalt thickness of the eastbound lanes ranged from 10 to 12 inches. Thickness of the stone base ranged from 4 to 5 inches. Thickness of the stabilized layer ranged from 8 to 16 inches. Asphalt thickness of the westbound lanes ranged from 11 to 12.5 inches and the thickness of the stone base ranged from 3 to 6 inches. The stabilized layer ranged from 8 to 16 inches in

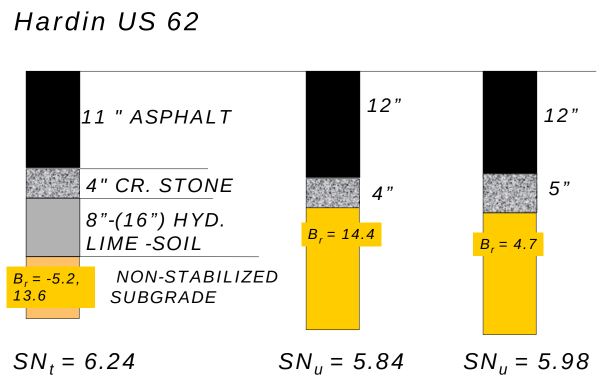

Figure 40. Pavement cross sections.

thickness. As noted above, the deeper areas of stabilized subgrade occurred when hydrated lime was used to dry wet areas of the soil subgrades.

In situ CBR-values of the untreated layer below the stabilized layer were much lower than the values of the stabilized layer, as shown in Tables 10 and 11. CBR-values of the non-stabilized subgrades of the eastbound lanes located below the stabilized layer averaged 6 (only 4.2 if one value is excluded) and 4.2 for the non-stabilized subgrade of the westbound lanes. CBR-values of the stabilized layers averaged 116

Table 10. US 62 Hardin County, Section 1, Eastbound lanes

MP	A. C. Thickness (in.)	Stone Base Thickness (in.)	Lime Stabilized Subgrade Thickness (in.)	Stabilized CBR	Non Stabilized CBR	SPT Blows per 6-in. intervals
13.75 EB	10.5	4.0	10	153.3	3.5	12/14/10
13.75WB	12.0	5.0	>16	185.5	13.6	8 / 3/ 5
13.95 EB	11.0	5.0	8	95.3	3.0	8/ 5/ 8
13.95WB	10.0	5.0	>16	96.8	5.2	8/ 4/ 5
14.20WB	10.0	4.0	15	49.3	5.2	15/ 7 / 5
14.50WB	12.0	4.0	None		14.4	4/ 6/ 7
14.60WB	12.0	5.0	None		4.7	2/ 1/ 2

Table 11. US 62, Hardin County, Section 2, Westbound lanes

MP	A. C. Thickness (in.)	Stone Base Thickness (in.)	Lime Stabilized Subgrade Thickness (in.)	Stabilized CBR	Non Stabilized CBR	SPT Blows per 6-in. intervals
12.00 EB	11.0	3.0	11	108.3	5.0	17/15/11
12.20WB	11.0	5.0	8	50.5	4.8	11 / 8/10
12.45WB	11.0	5.0	8	157.0	2.5	14/ 5/ 7
12.50 EB	11.0	4.0	16	59.8	11.8	23/28/28
12.80WB	12.5	6.0	10	95.3	2.0	25/13/13
12.90 EB	11.0	4.0	8	49.0	1.4	12/ 6/ 6
13.70WB	11.0	4.0	16	103.0	1.9	15/14/10

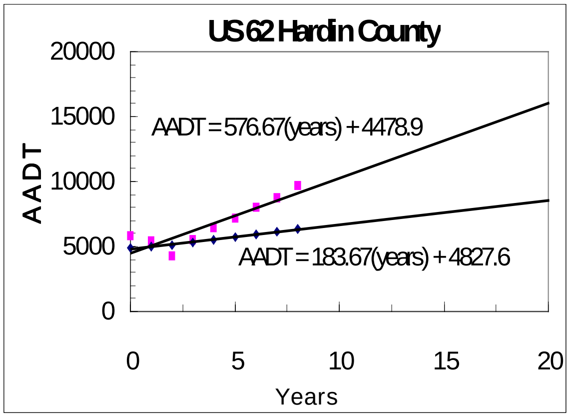

Figure 41. Relationship between AADT and time.

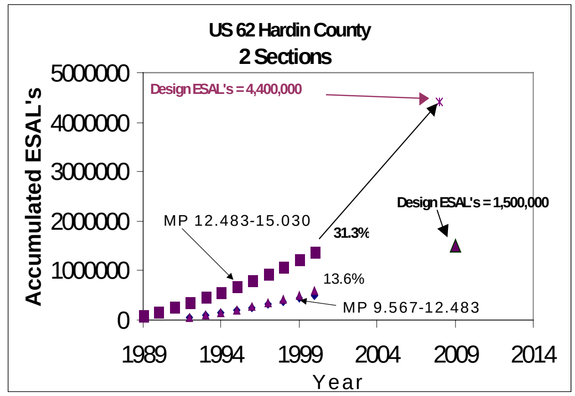

Figure 42. Accumulated ESALs as a function of year.

for the eastbound subgrade while the CBR-value of the westbound lanes averaged 89. The CBR values of the stabilized layer were some twenty times greater than the CBR values of the non-stabilized subgrade.

The growths in AADT of the two different sections are shown in Figure 41. After 3 years, the AADT ranged from about 4910 to 5810. After 11 years these values had growth to a range of 6360 to 9610. Estimated ranges of AADT for 15 and 20 years are 7,580 to 13,129 and 8500 to 16,100, respectively. One section had a design life of 20 years and 4,400,000 ESALs. The other section of US 62 in Hardin County had a design life of 20 years and 1,500,000 ESALs. Estimated values of ESAL are shown in Figure 42. After 11 years, about 14 to 31 percent of the design ESALS at this site have occurred.

Rideability index as a function of time is shown in Figure 43. Initially, the RI-values of the two sections were about 3.8. After 8 years, the values had decreased to only 3.60 to 3.67. Projected values of RI, based on the equations in Figure 43, after 15 and 20 years are 3.64 to 3.57 and 3.62 to 3.55, respectively.

Average rutting depths at the 50th and 20th percentile test values, Figure 44, are 0.22 and 0.28 inches, respectively.

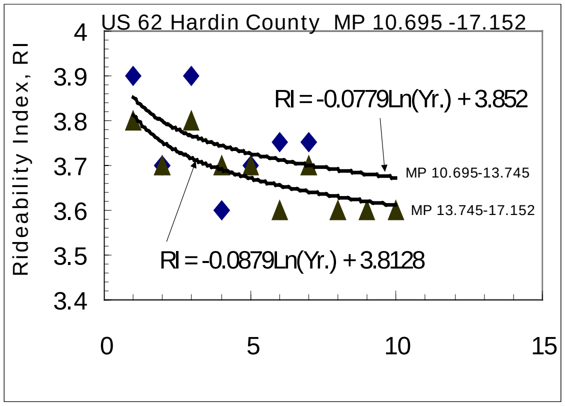

Figure 43. Rideability Index as a function of time for US 62 in Hardin County.

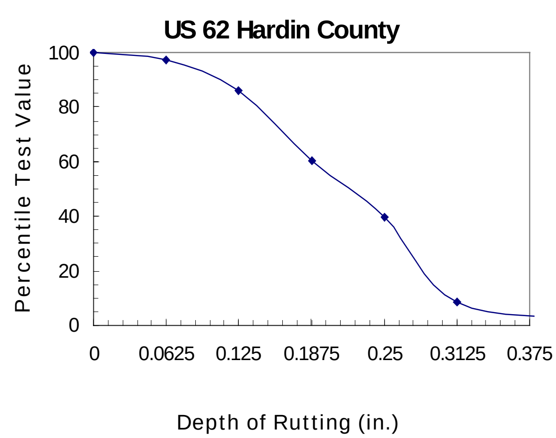

Figure 44. Depth of rutting as a function of percentile test value.

Owen County, US 127

This portion of US route 127 was reconstructed in 1990 and begins at about MP14.3 and ends about 0.1 mile south of the intersection of this roadway and KY 22, MP 15.4. Record plans of the Kentucky Transportation Cabinet show that the project starts at station 932+ 50 and ends at station 982 +50. Five percent of hydrated lime (by dry weight) was recommended for stabilization. The recommended depth is not known.

Table 12. Drilling results of the Owen County site, US 127.

MP	A. C. Thickness (in.)	Crushed Stone Base Thickness (in.)	Lime Stabilized Subgrade Thickness (in.)	Stabilized CBR	Non Stabilized CBR	SPT Blows per 6-in. intervals
14.3 NB	11.5	4.0	8.0	30.0	5.2	6/ 5/ 8
14.7 NB	9.5	4.0	5.0	44.5	3.2	6/ 6/ 6
15.1 NB	8.5	4.0	5.0	26.7	2.9	4/ 2/ 5
15.3 SB	8.5	5.0	8.0	59.4	4.4	9/ 4/ 5
14.5 SB	9.5	4.0	11.0	110.3	None *	16/10/50
14.3 SB	10.0	4.0	8.0	None	None	9/ 6/ 7

* Rock: 11 inches below top of subgrade

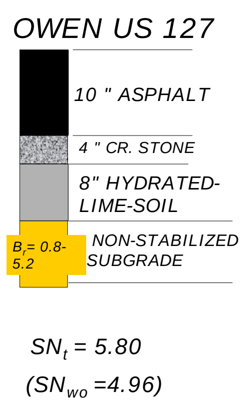

Figure 45. Pavement cross section.

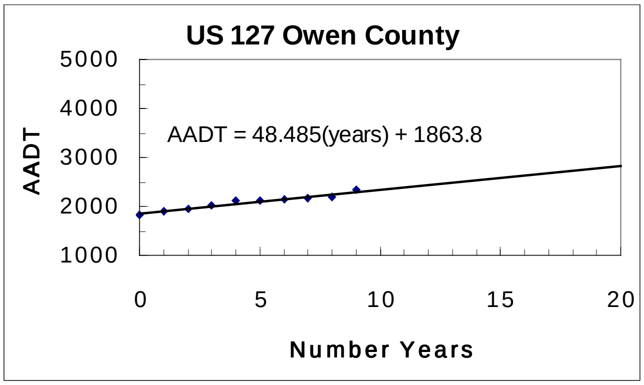

Figure 46. AADT as a function of time.

Drilling results are shown in Table 12. A typical cross of this stabilized roadway section is presented in Figure 45. Thickness of the asphalt layer ranged from 8.5 inches to 11.5 and averaged 9.5 inches. Except for one location , the stone base thickness was 4 inches. Thickness of the hydrated lime-soil layer ranged from 5 to 11 inches and averaged about 7.4. CBR-values of the stabilized layer ranged from 27 to 110 and averaged 54. Values of CBR of the untreated layer located below the treated layer ranged from 2.9 to 5.2 and averaged only 3.9. The CBR strength of the stabilized layer was about 14 times greater than the untreated layer.

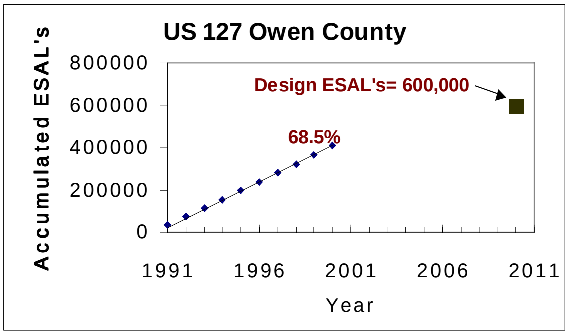

Figure 47. Accumulated values of ESALs.

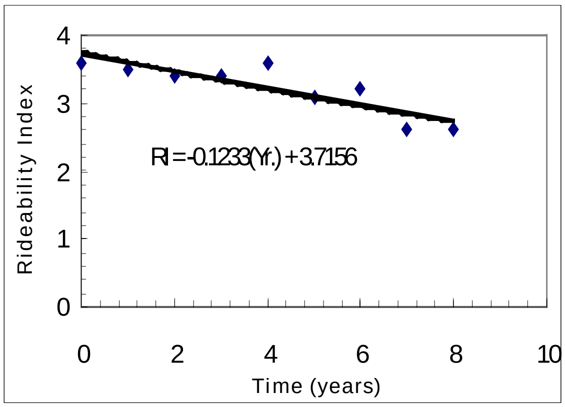

Figure 48. Rideablity index as a function of time.

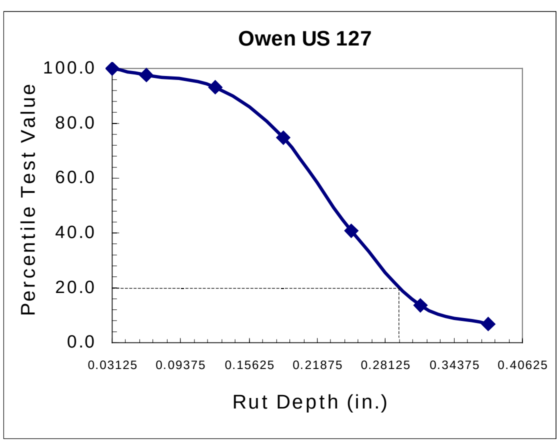

Figure 49. Percentile test value as a function of time.

The section had a design life of 20 years and 600,000 ESALs. Values of AADT as a function of time are given in Figure 46. After 9 years, the AADT-value has increased from about 1860 to 2300. Projected values of AADT for 15 and 20 years are 2590 and 2830, respectively. An estimated value of accumulated ESALS at the end of 9 years is 411,000, as shown in Figure 47. About 69 percent of the design life (600,000 ESALS) of this pavement has occurred.

Rideability index as a function of time is shown in Figure 48. After 8 years, the RI-value is 2.73. At the end of 15 and 20 years, the projected RI-values are 1.87 and 1.67, respectively.

The relationship between percentile test value and depth of rutting is shown in Figure 49. At the 20th percentile test value, the depth of rutting is 0.29 inches.

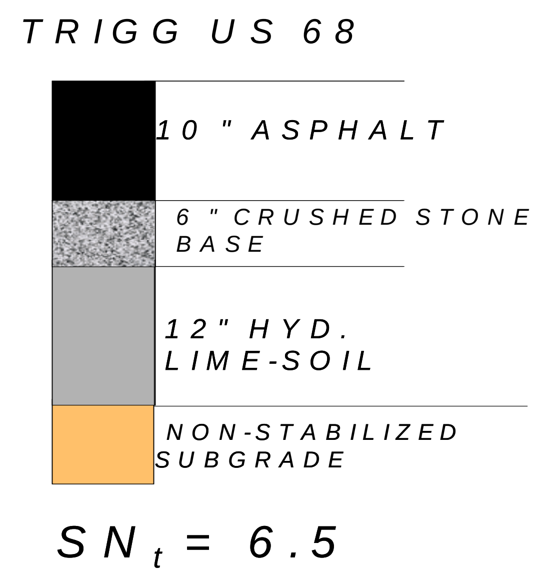

Figure 50. Cross section.

Trigg County, US 68, West Bound Lane (only)

The subgrade of this site was stabilized during the reconstruction of existing US 68 and KY 80 in 1993. The project began at the intersection of US 68 with KY route 3468, MP 20.96, and ends at the intersection of US 68 with I-24, MP 24.4.

Table 13. Summary of drilling results, Trigg County, US 68.

MP	A. C. Thickness (in.)	Crushed Stone Base Thickness (in.)	Lime Stabilized Subgrade Thickness (in.)	Stabilized CBR	Non Stabilized CBR	SPT Blows per 6-in. intervals
21.5	9.0	5.0	12.0	46.3	1.9	6/ 7/ 6
22.1	10.0	6.0	11.0	44.5	3.0	21/15/ 9
22.7	9.0	6.0	10.0	184.4	9.2	16/14/10
23.0	9.0	5.0	8.0	98.5	7.5	9/ 4/ 5
23.6	10.0	5.0	11.0	29.3	8.3	11/ 9/ 8
24.2	10.0	5.0	12.0	147.0	6.0	14/23/49

Samples obtained during field testing and sampling indicated that the depth of stabilized subgrade was 8 to 12 inches. Five percent (by dry mass) of hydrated lime was recommended for performing the subgrade stabilization. Limits of the project stations, as shown by the record plans of the Kentucky Transportation Cabinet, extended from Station 338+00 to Station 538 +50.

Drilling results are presented in Table 13. Thickness of the asphalt layer ranged from 9 to 10 inches and averaged 9.5. Thickness of the crushed stone base ranged from 5 to 6 inches. Thickness of the stabilized layer ranged from 8 to 12 inches and averaged 10.7 inches. A cross section of the pavement is shown in Figure 50. Values of CBR of the stabilized layer ranged from 29 to 184 and averaged 92. CBR values of the untreated subgrade below the treated layer ranged from 1.9 to 9.2 and averaged 6. The CBR strength of the stabilized layer was about 15 times the CBR strength of the untreated subgrade.

The assumed design life of this section of US 68 years was 20 years and the assumed accumulated value of ESAL at the end of that design life was 3,400,000. About 33 percent of the design ESAL value has occurred. The relationship between AADT and time is shown in Figure 51. Initially, the AADT was about 8200. At the end of 7 years, the value gradually increases to 9590. Projected AADT values after 15 and 20 years are 11,155 and 12,137, respectively. In Figure 52, the estimated accumulated ESALs are shown as a function of time.

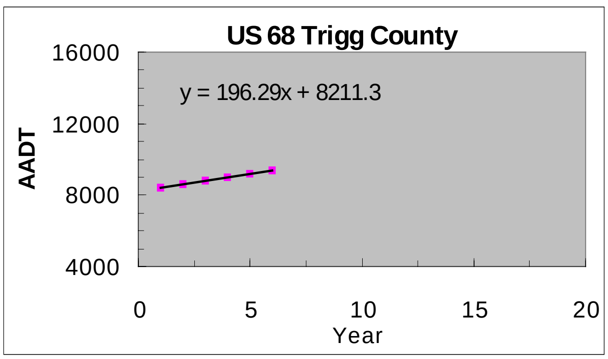

Figure 51. AADT as a function of time.

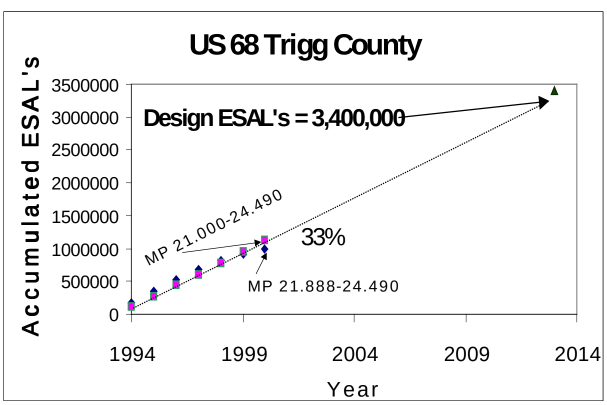

Figure 52. Accumulated ESALs as a function of time.

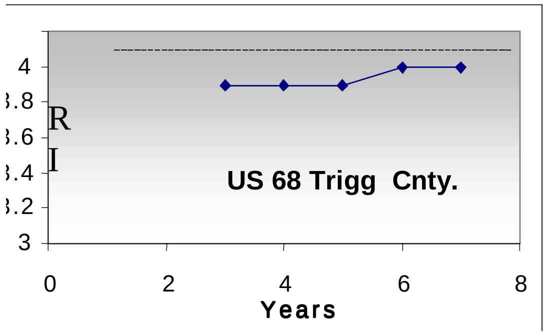

Figure 53. Rideability indices.

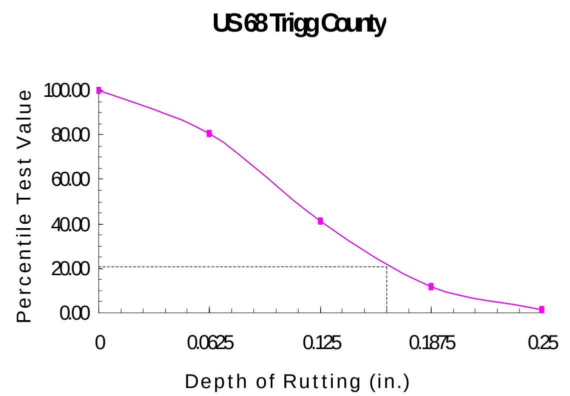

Figure 54. Depth of rutting.

The relationship between rideability index and time is shown in Figure 53. RI-values range from 3.9 to 4.0 for this site. Depth of rutting, as a function of percentile test value, is shown in Figure 54. At the 20th percentile test value, the rut depth is about 0.163 inches.

Boone County, KY 842

This stretch of KY 842 route was constructed in 1987 and 1988 as a connector road between US routes 25 and 42. A bridge crossing Interstate 75 is located in the section. The route was originally designated as KY route 1018. The section of road between the west-end of the I-75 Bridge and US route 42 (MP 1.18 to 2.57) was constructed in 1987. The subgrade was stabilized with 10 percent (by dry mass) of Portland cement and had a design depth of 12 inches. The contractor experienced difficulties mixing the cement with the clay subgrade. Clay clods formed during the mixing operation because the cement did not penetrate the clay clods very well. A recommendation was made to stabilize the remainder of the subgrade with a combination of hydrated lime and Portland cement.

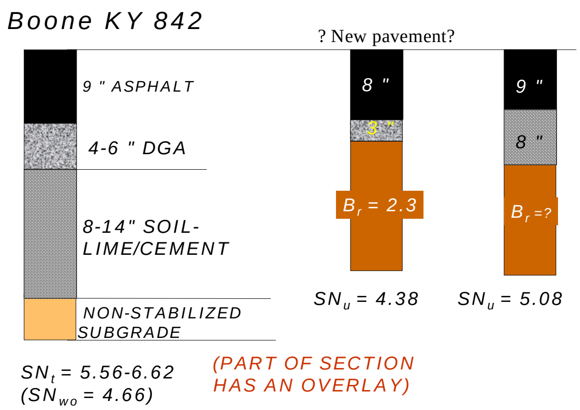

Figure 55. Cross Sections.

This second section, which begins at the intersection with US Route 25 and ends at the east end of the I-75 bridge (MP 0.0 – 1.105), was constructed in 1988. The subgrade was stabilized by initially mixing three percent (dry mass) hydrated lime with the existing soil. After mixing the lime and soil, a 48-hour curing period was specified to allow the hydrated lime, and water used during mixing, to mellow or break down clay clods. Hydrated lime is generally very efficient in penetrating and breaking down the clay clods that form during mixing. Portland cement (four percent by dry weight) was mixed with the hydrated lime-soil mixture within 72 hours following the preliminary curing period. The hydrated lime reacts with the clay particles and usually the clay particles are transformed into silty size particles. Once this occurs, the cement can penetrate and react with the silty particles

Table 14. Drilling Results, Boone County, KY 842.

MP	AC Thickness (in.)	Crushed Stone Drainage Base Thickness (in.)	Stabilized Subgrade Thickness (in.)	Stabilized CBR	Non Stabilized CBR	SPT Blows per 6-in. intervals
0.20	7.0	5	12[1]	50.8	5.8	14/14/11
0.55	9.0	5	11[1]	15.3	3.2	6 / 3/ 4
0.95	9.0	6	14[1]	78.6	3.5	16/16/10
1.35	9.0	3	10[2]	85.7	2.7	7 / 13/ 6
1.70	9.5	4	8[2]	36.5	3.0	15/ 4/ 4
2.10	8.0	3	None		2.3	3 / 5 / 4
2.30	9.0	None	8[2]	NA	NA	11/ 8/ 5

1. Hydrated lime treatment
2. Hydrated lime-Portland cement treatment

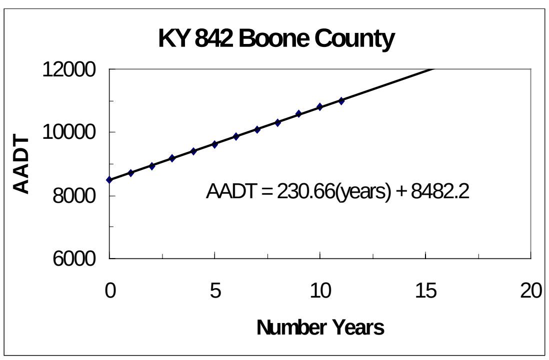

Figure 56. AADT-time relationship.

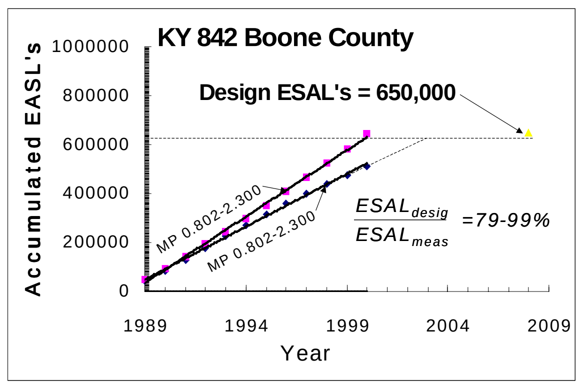

Figure 57. Value of ESALs as a function of year.

to create good bonding between particles. The lime-cement-soil subgrade was compacted within four hours after mixing with cement[3]. The pavement design consisted of 8 inches of asphalt and 5 inches of crushed stone base. A typical cross section is shown in Figure 55. Results of field borings are summarized in Table 14. Thickness of the asphalt layer ranged from 7 to 9.5 inches and averaged 8.6 inches. Thickness of the crushed stone base varied from 3 to 6 inches and averaged 3.8 inches. The stabilized layer consisted of a mixture of hydrated lime and soil and ranged in thickness from 8 to 14 inches. Thickness of the stabilized layer averaged about 10.5 inches. In one boring, a stabilized layer was not present while in another boring no crushed stone base was found. CBR values of the stabilized layer ranged from 15 to 79 while the CBR values of the untreated layer below the treated layer ranged from 2.3 to 5.8. Average CBR values for the stabilized layer and the untreated layer were 53 and 3.4, respectively. Bearing strength of the stabilized layer was about 16 times greater than the CBR value of the untreated subgrade.

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[3] (Memorandum C-5-88: Kentucky Transportation Cabinet).

The design life of the section was 20 years. After 11 years, the AADT (Figure 56) was about 11,000. Initially, the AADT was 8,482. Estimated values of AADT after 15 and 20 years are 11,944 and 13,095, respectively. An accumulated design ESAL value of 650,000 was assumed for the 20-year design period. A computer program used to predict ESALs (Rister and Allen 1999) showed that the accumulated ESALs were equal to the design EASLs by 1999, as shown in Figure 57.

Consequently, an overlay was placed on the section from MP 0.0 to about MP 1.9, and from approximate MP 2.1 to 2.45 in September or October of 1999. Two small sections of pavement stretching from approximate MP 1.9 to 2.0 and from 2.45 to 2.572 did not appear to be overlaid at the time of this study. They had been overlaid, or patched, by the end of the study, possibly due to residential and commercial development along the roadway.

The decrease in the rideability index with increasing time is presented in Figure 58. At the end of 10 years the estimated RI–value was 3.27. At the end of 15 and 20 years, estimated values are 3.21 and 3.18, respectively. Since the pavement had been overlaid, rutting measurements were not obtained.

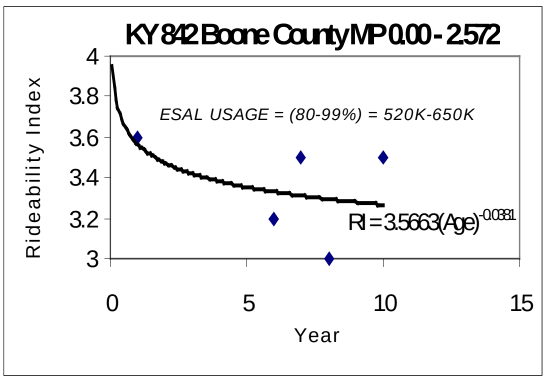

Figure 58. Rideability index as a function of time.

McCracken County, US 62

Coring and sampling indicated that the pavement at this site was constructed with 7 to 8.5 inches of asphalt resting on 4 inches of dense graded aggregate base. The section begins at MP 8.8 and ends at MP 10.015. Bank run gravel, ranging from 9 to 21 inches in thickness, was used as a subbase. The subgrade was stabilized with lime kiln dust from about MP 9 to the end of the section, MP 10.015. Thickness of the soil-lime kiln dust layer ranged from 10 to 13 inches. From the beginning of the project at MP 8.8 to MP 9, no stabilization was used. Standard Penetration Tests (SPT) at this site began at the top of the bank gravel subbase. Cross sections recorded at the site are presented in Figure 59.

Drilling results are shown in Table 15. In situ CBR values of the soil subgrade stabilized with kiln dust ranged from 12 to 49 while CBR values of the bank gravel ranged from 13 to 73. Average CBR values of the lime kiln dust-treated layer and the bank gravel were 32 and 36, respectively. In situ values of CBR of the untreated soil subgrade varied substantially and ranged from about 7 to 73.

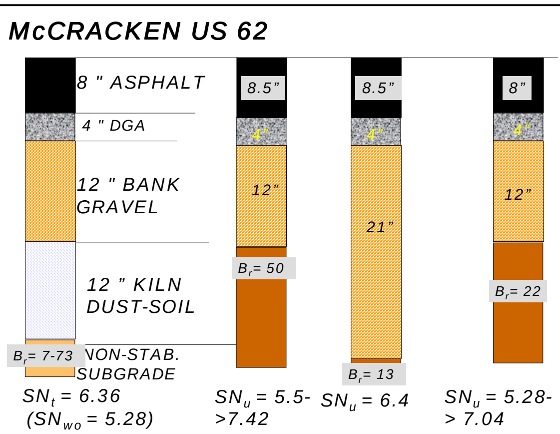

Figure 59. Cross sections.

The relationship between AADT and time is shown in Figure 60. The AADT-value, after 10 years, has increased from an initial value of about 6,675 to 9,320. Estimated values of AADT after 15 and 20 years are 10,650 and 11970, respectively. The value of ESAL as a function of time is shown in Figure 61. The section had a design life of 20 years and 2,000,000 ESALs. After 10 years, an estimated 23 to 43 percent of the design ESALs has occurred.

Table 15. Drilling results, McCracken County, US 62

MP	A. C. Thickness (in.)	Gravel Base Thickness (in.)	Lime Kiln Dust Stabilized Subgrade Thickness (in.)	Stabilized CBR	Bank Gravel CBR	SPT Blows per 6-in. intervals
8.8	8.5	4.0 in. DGA 12.0 in. Bank Gravel	None		50.3	17/12/ 7
8.95	8.5	4.0 in DGA 21.0 in. Bank Gravel	None		13.0	11/19/23
9.13	7.0	5.0 in DGA 10.0 in. Bank Gravel	12.0	23.8		6/ 7/ 17
9.32	8.0	4.0 in DGA 12.0 in. Bank Gravel	12.0	12.3		12/11/16
9.51	8.0	4.0 in DGA 12.0 in. Bank Gravel	10.0		22.3	10/11/13
9.72	8.0	4.0 in DGA 10.0 in. Bank Gravel	13.0	42.3	72.5	18/23/20
9.95	8.0	4.0 in DGA 9.5 in. Bank Gravel	12.0	49.3	24.5	7/20/55

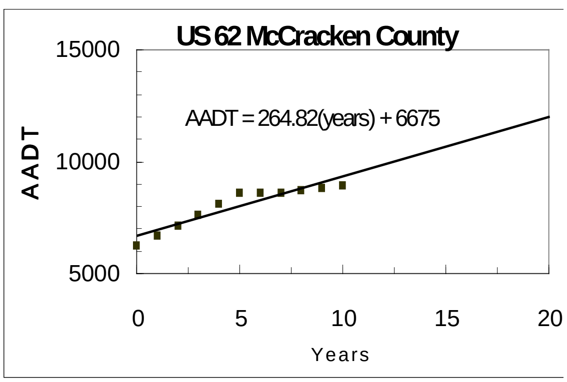

Figure 60. AADT-time relationship.

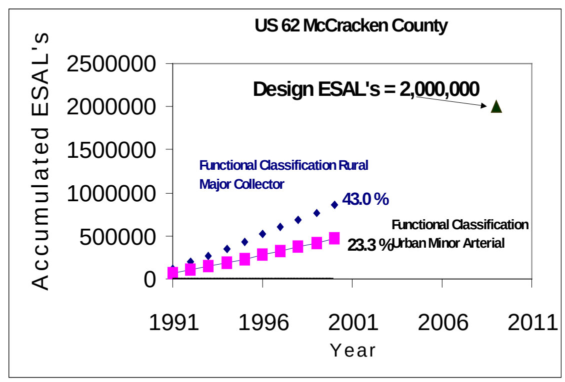

Figure 61. ESALs as a function of time.

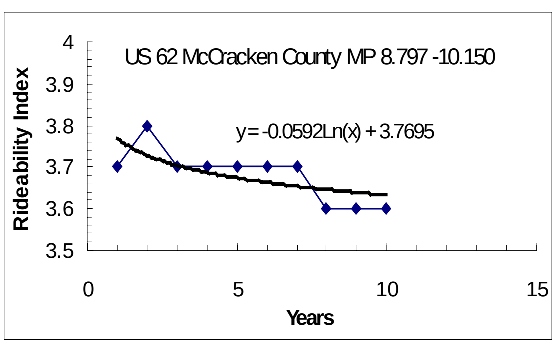

Figure 62. Trend of rideability index and time.

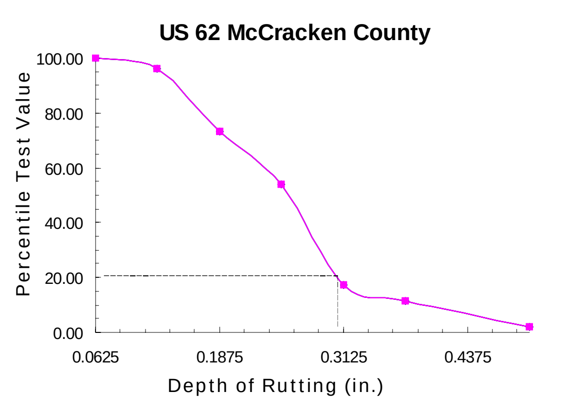

Figure 63. Depth of rutting.

Hickman County, US 51

This section of US 51 is a two-lane roadway that begins at MP 12.8 and ends at MP 14.45. Two bridges are located within the section at MP 13.055 and 13.326. The roadway was realigned when the two bridges were constructed in 1990. Lime kiln dust was used to stabilize wet, silty soils encountered during construction. A typical cross section is shown in Figure 64.

Drilling results are presented in Table 16. Thickness of the asphalt pavement components ranged from 8.5 to 10.0 inches. Thickness of the base aggregate ranged from 5.0 to 7.0 inches. Thickness of the stabilized lime kiln dust-subgrade layer ranged from 8 to 16 inches. The wide range of thicknesses could be attributed to the fact that lime kiln dust was used to dry the subgrade soils, or remove excess moisture. After 10 years, the CBR values of the stabilized layer ranged from 18 to 58 and averaged 38. CBR values of the untreated soils ranged from 2 to 9.6 and averaged about 5.6. Bearing strengths of the stabilized layer was almost seven times the bearing strengths of the untreated subgrade soils.

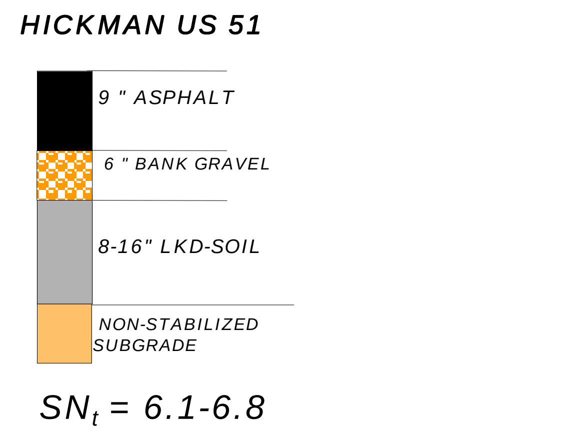

Figure 64. Cross Section.

Table 16. Drilling results, Hickman County, US 51.

MP	A. C. Thickness (in.)	Bank Gravel Base Thickness (in.)	Lime Kiln Dust Stabilized Subgrade Thickness (in.)	Stabilized CBR	Non Stabilized CBR	SPT Blows per 6-in. intervals
12.9	10.0	6.0	11.0	18.3	5.1	8/ 7/ 4
13.15	9.0	5.0	8.0	58.0	NA	15/18/60
13.25	9.0	5.0	8.0	40.0	9.6	8/ 4/ 5
13.43	9.0	6.0	16.0	17.5	2.0	6/ 1/ 1
13.6	8.5	7.0	10.0	43.3	6.9	26/42/33
13.81	9.0	6.0	8.0	49.8	4.5	23/12/14

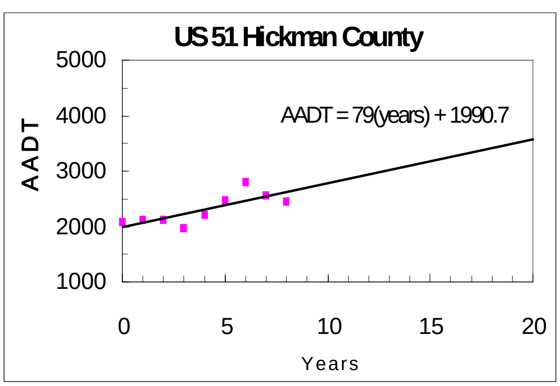

Figure 65. AADT as a function of years.

Values of AADT are shown in Figure 65 as a function of time. The section had a design life of 20 years and 1,300,000 ESALs. The initial value of AADT was about 1990. After 8 years that value had grown to 2,622. Projected values of AADT after 15 and 20 years are 3,176 and 3,571, respectively. Estimated values of accumulated ESAL are shown in Figure 66. Approximately, 32 percent of the design ESALs has occurred.

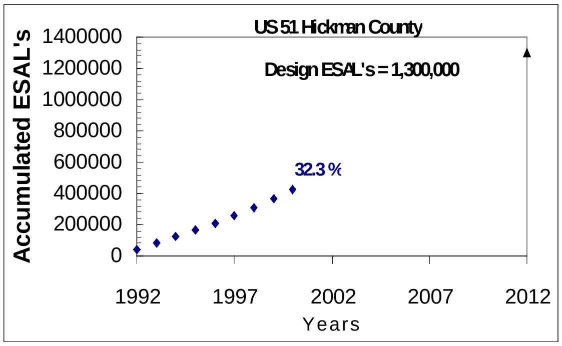

Figure 66. Estimated accumulated ESALs as a function of years.

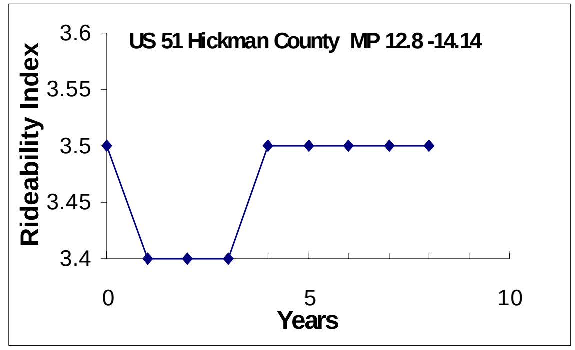

Figure 67. RI as a function of years.

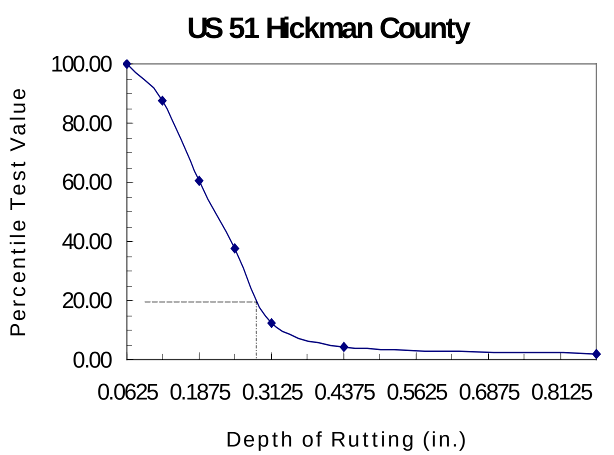

Figure 68. Depth of rutting.

Rideability index of the site ranges from 3.4 to 3.5, as shown in Figure 67. The last five measured values of RI at the site are equal to 3.5.

Average rutting measurements are shown in Figure 68. At the 20th percentile test value, the average rutting depth is 0.28 inches.

Breckinridge County, US 60

This section of roadway begins at MP 12.929, the intersection of KY 992, and ends at MP 16.391. The subgrade was stabilized with five percent Portland cement to depths ranging from 12 to 18 inches. Five locations were tested within the project limits. However, only three locations were stabilized as shown below, in Table 17. One test location at MP 16.8 was situated just beyond the stabilized section. This section of US route 60 was reconstructed in 1987. Pavement cross sections of some of the borings are shown in Figure 69.

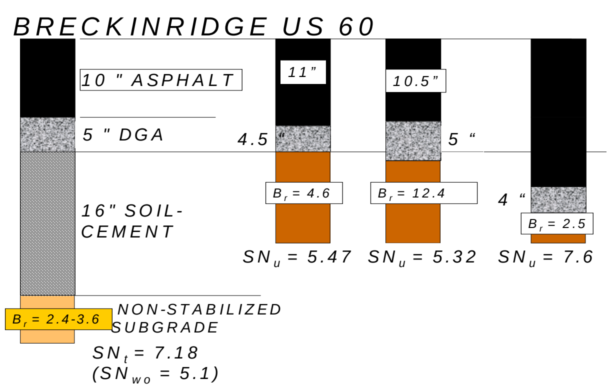

Figure 69. Pavement cross sections of borings.

At 5 of 6 boring locations in the non-stabilized subgrade, the in situ CBR ranged from 2.4 to 4.6. At one location the value was 12.4. CBR values of the stabilized sections ranged from 58 to 107. The average bearing strength of the stabilized subgrade was about 16 times greater than the average bearing strength of the non-stabilized subgrade.

The study section had a design life of 20 years and 1,900,000 ESALs. In Figure 70, AADT is presented as function of time. Initially, the AADT was about 1770. After thirteen years, the

AADT has increased to 3,296. At 15 and 20 years, projected values of AADT are 3,530 and 4,118, respectively. As shown in Figure 71, the estimated accumulated ESALS range from 38 to 118 percent of the design ESAL. The rideability index after about eleven years is 3.69, as shown in Figure 72. Projected values of RI at 15 and 20 years are 3.66 and 3.64, respectively. Average depth of rutting at the 20th percentile test value is about 0.32 inches, as shown in Figure 73.

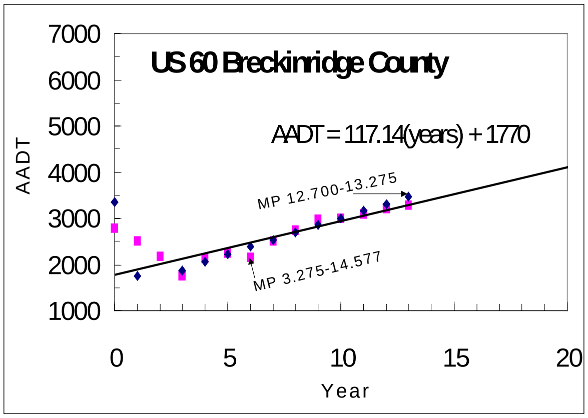

Figure 70. AADT as a function of years.

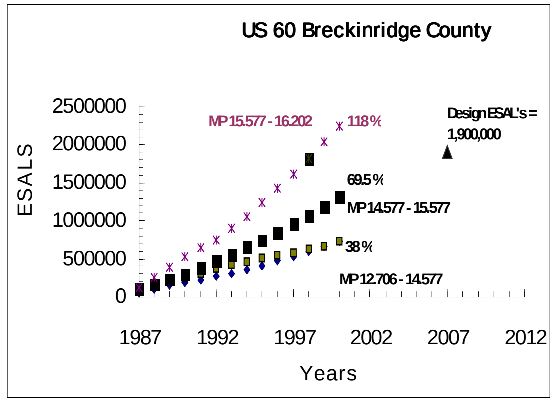

Figure 71. ESALs as a function of time.

Table 17. US 60 Breckinridge County

MP	A. C. Base Thickness (in.)	DGA Base Thickness (in.)	Cement Stabilized Subgrade Thickness (in.)	Stabilized CBR	Non Stabilized CBR	SPT Blows per 6-in. intervals
13.2	11.5	5.0	16	106.5	2.4	40/61/24
13.8	11.0	4.5	None		4.6	9 / 5/ 6
14.3	10.5	5.0	None		12.4	4 / 7/ 10
15.2	11.0	5.0	14	61.8	3.6	27/24/17
15.9	10.0	3.0	16	57.5	3.2	25/41/21
16.8	30.0	5.0	None		2.5	5/ 6/ 9

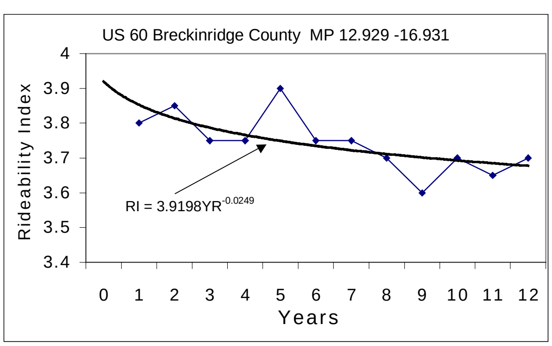

Figure 72. RI as a function of time.

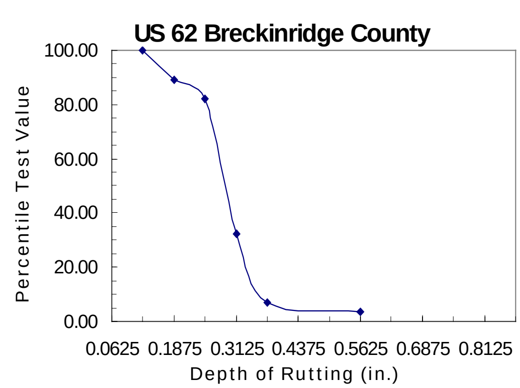

Figure 73. Depth of rutting.

Daviess County, KY 331, River Port Access Road

The section evaluated begins at MP 0.5 and ends at MP 1.54. It was constructed in 1986. It was originally called the River Port Access Road. Sections of the stabilized and non-stabilized roadway are shown in Figure 74. In the non-stabilized areas, asphalt patching thickness ranges up to some 30 inches. No patching was encountered in the stabilized area.

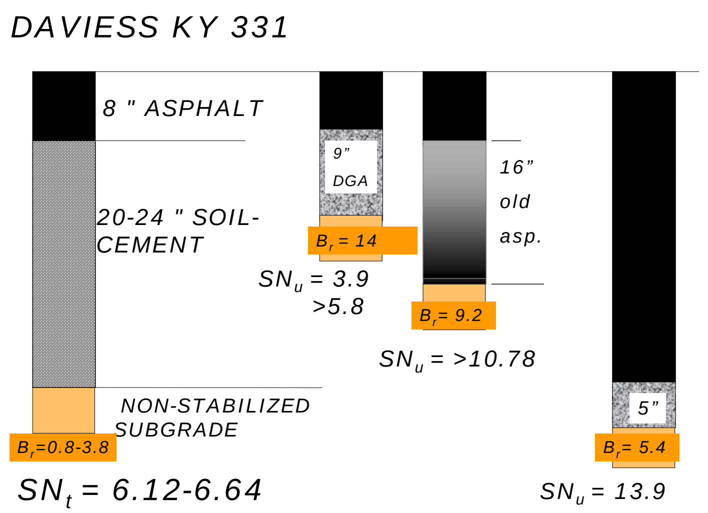

Figure 74. Cross sections.

Portland cement was used to dry the excess moisture in the subgrade soils at this site. Six percent (by dry weight) of Portland cement was used to stabilize the soils. Results of field borings are summarized in Table 18. CBR values of the stabilized subgrade ranged from 81 to 90 while values of CBR of the non-stabilized subgrade ranged from 0.8 to 14. At three locations, the values ranged from 0.8 to 3.8. Bearing strengths of the cement-treated soils were some 6 to 112 times greater than the strengths of the untreated soils.

AADT as a function of time is shown in Figure 75. The AADT after some 13 years in service was 6,591. Projected values of AADT at the end of 15 and 20 years are 6,818 and 7,382, respectively. Estimated accumulated ESALs as a function of time are shown in Figure 76. The section had a design life of 20 years and 1,000,000 ESALs. After 14 years, more than 72 percent of the design life had occurred. The section was overlaid in 1999 after field tests at this site had been completed. RI-values for this site were not available. Field measurements of rutting depths were not obtained because of the recently constructed overlay.

Table 18. KY 331, Daviess County

MP	A. C. Base Thickness (in.)	DGA Base Thickness (in.)	Cement Stabilized Subgrade Thickness (in.)	Stabilized CBR	Non Stabilized CBR	SPT Blows per 6-in. intervals
0.8	6.0	9.0	No Stab.		14.0	8 /12/15
0.9	9.0	None	20.0	88.8	3.8	50/ /
1.0	8.0	None	22.0	90.0	3.0	50/ /
1.1	8.0	None	24.0	81.2	0.8	50/ /
1.2	8.5	16.0 (old AC ?)	No Stab.		9.2	6/ 6/ 7
1.5	30.0	5.0	No Stab.		5.4	6/ 6/ 5

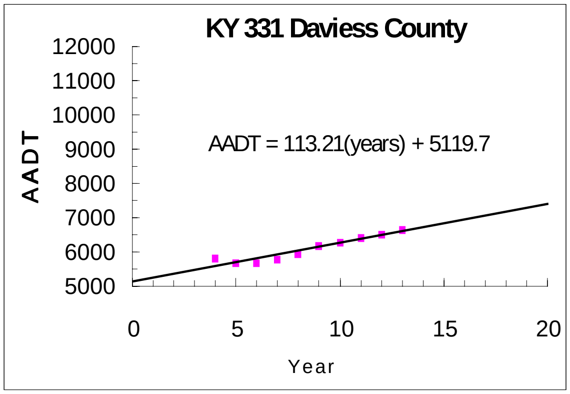

Figure 75. AADT as a function of time.

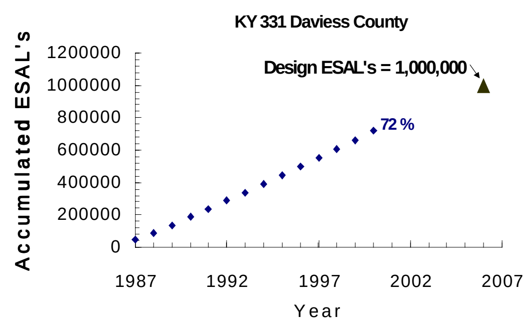

Figure 76. Estimated accumulated ESALs.

INDEX PROPERTIES OF STABILIZED AND NONSTABILIZED SUBGRADES

Soil index properties of subgrades mixed with chemical admixtures and non-stabilized soil subgrades were determined. These tests included liquid and plastic limits, grain-size analysis, specific gravity, and soil classifications. Each specimen was classified using the Unified and AASHTO Soil Classification Systems. The data are summarized in Appendix C, Tables C-1 through C-4. A sufficient number of soil samples were collected during the field operations so that a comparison could be made between the index properties of the untreated subgrade soils and the index properties of the subgrade soils after mixing with chemical admixtures. Results of the grain-size analysis are presented in APPENDIX D, Tables D-1 through D-3. The percent finer than the U.S. No. 200 sieve (0.075 mm) and 0.002 mm-size particle size are summarized for both the untreated and chemically treated subgrade soils. In situ moisture contents measured at the tops and points below the tops of the untreated and treated subgrades are summarized in APPENDIX E, Tables E-1 to E-8.

RESILIENT MODULUS OF UNTREATED AND TREATED SOIL SUBGRADES

Resilient modulus has been proposed as a means of characterizing the elastic properties of pavement materials. It is expressed as the ratio of deviator stress applied to the soil and the resilient axial deformation recovered after release of the deviator stress. The assumptions are made that pavement materials are designed for loading in the elastic range and that the resilient modulus is the only parameter needed to design the thickness of a pavement. Although empirical relations have been used in the past to estimate the resilient modulus of soils, the trend in recent years is to measure the resilient modulus of soils (and other pavement materials) using laboratory tests. Empirical relations attempt to relate the resilient modulus to some type of soil parameter, such as bearing ratio (CBR), or resistance index (R_{value}). A fundamental problem with empirical relations is the models attempt to assign a fixed value of resilient modulus to a given type of soil. However, the value of resilient modulus is stress-strain dependent, that is, the value changes as stress and strain conditions change. In recent years, the resilient modulus testing procedure for soils and aggregates has steadily evolved and become a standard testing method of the American Association of State Highway and Transportation Officials (2000). This testing standard is referred to as AASHTO T 292-91.

Equipment for performing resilient modulus tests of soils and aggregates has steadily evolved and improved over the past few years.

Several mathematical expressions are available for modeling the resilient modulus of soils and aggregates. These include models proposed by Moossazadeh and Witczak (1981), Dunlap (1963), Seed et al (1967), May and Witczah (1981) and Uzan (1985). The effectiveness of these models to relate the resilient modulus to stresses is examined herein. Difficulties are encountered in using these models because they are not too effective in considering the effects of both the confining stress and deviator stress on the resilient modulus of soils. To correctly model the resilient modulus of soils, a new model has been proposed by Ni et al (2002). Resilient modulus tests were performed on the untreated and chemically treated subgrade specimens obtained from the field. Resilient modulus data obtained from testing the field specimens were analyzed and compared using various published models, including a newly proposed model.

Sampling

Resilient modulus tests were performed on "undisturbed" specimens of the subgrades treated with chemical admixtures. Treated, undisturbed specimens of soil-hydrated lime, soil-cement, soil-kiln dust, and soil-AFBC were tested. Also, the tests were performed on undisturbed specimens of the untreated subgrade. Specimens of treated subgrades obtained from thin-walled sampling tubes generally were of low quality because they were usually very brittle after extrusion from the tube. Tube samples could usually be obtained from soil-hydrated lime, kiln dust, and soil-AFBC subgrades. Tube samples could not be obtained from soil-cement subgrades. Although resilient modulus tests were performed on several tube specimens of treated subgrades, the data were not included in this report because of the poor quality of the specimens after extrusion. To obtain good quality specimens of treated subgrades, a drilling technique was perfected during the field operations. In this technique, high volume air pressure is used as the "drilling media" to avoid using water, which would have destroyed the integrity of the in situ specimen. Consequently, good quality chemically treated specimens were obtained for resilient modulus testing. Although quality, thin-walled tube specimens of chemically treated subgrades could not be obtained, good quality, thin-walled tube specimens of untreated subgrades could easily be obtained for resilient modulus testing.

Testing Equipment

The resilient modulus testing equipment located at the University of Kentucky Transportation Center is a model RMT-1000, obtained from Structural Behavior Engineering Laboratories, of Phoenix, Arizona. The system consists of a pressure control panel, triaxial cell, a hydraulic power supply, and computer and software for controlling the testing of a resilient modulus specimen. A view of the equipment is shown in Figure 77. The top and base of the triaxial cell are stainless steel and the chamber is acrylic plastic. The system is a complete, closed-loop, servo hydraulic triaxial testing system. Measurement transducers (load and displacement) are mounted internally in the acrylic triaxial chamber. Various load forms of different shapes are available for applying loading sequences by computer software. Computer software is used to record and reduce all data. A load actuator, Figure 78, applies the repeated loads. The load is applied for 0.1 second and released for 0.9 second. Details of this equipment have been given elsewhere (Hopkins et al 2002). The entire system is calibrated, or checked, periodically for performance. Also, resilient modulus tests are performed periodically on preformed rubber specimens. Initial values of resilient modulus of those specimens were established when the equipment was initially made operative.

System Components

The servo controller is a Model 547-1 with dual AC/DC feedback signal conditioning for load and deformation transfer. The signal conditioning system is a series 5 model 300 4-channel for 2 internal LVDT's and 2 pressure transducers. A view of the LVDTs mounted internally, on the sides of a specimen is illustrated in Figure 79. A load cell is mounted at the base of the specimen in the triaxial chamber.

The LVDT transducer calibrator is a Model 139 with 1-inch travel range and 0.00005 in resolution. The load cell, pressure transducer, and pore pressure transducer are calibrated using shunt calibration with preset resistance.

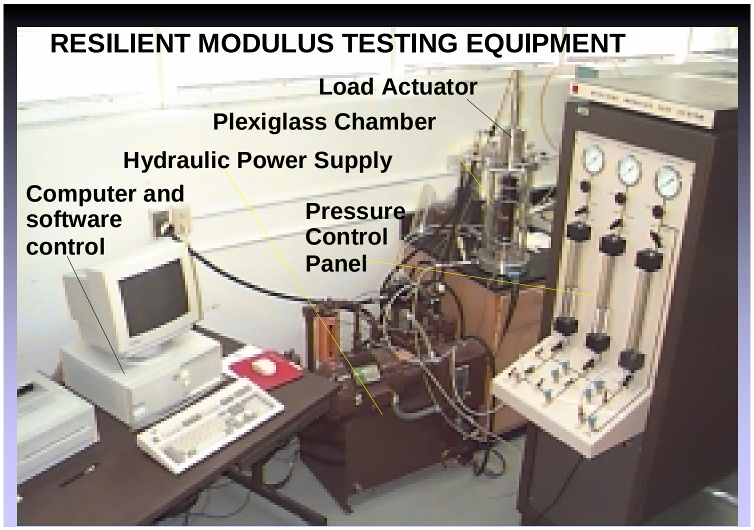

Figure 77. View of resilient modulus testing equipment.

Test Data

Eighty-nine resilient modulus tests were performed on specimens collected from the stabilized and non-stabilized subgrades. The tests were conducted according to AASHTO T-294 (2000). Confining stresses of 6, 4, and 2 psi (41.4, 27.6, and 13.8 kPa) and deviator stresses 10, 8, 6, 4, and 2 psi (68.9, 55.1, 41.4, 27.6, and 13.8 kPa) were used. One hundred conditioning cycles were used. Test data are summarized in Tables F-1 through F-20.

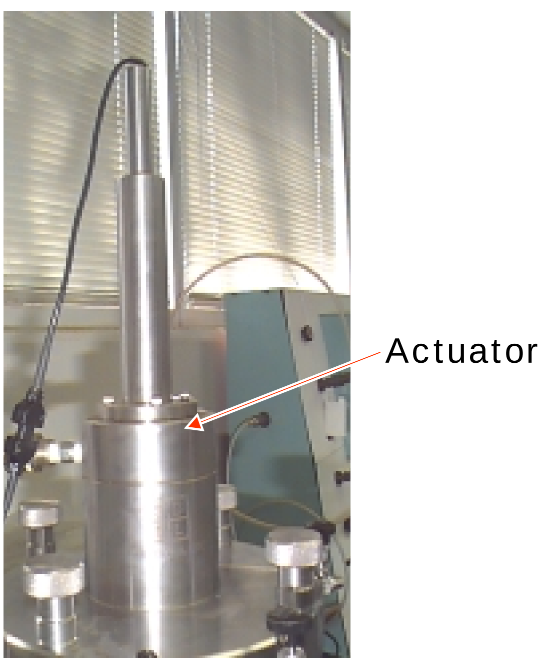

Figure 78. View of loading actuator.

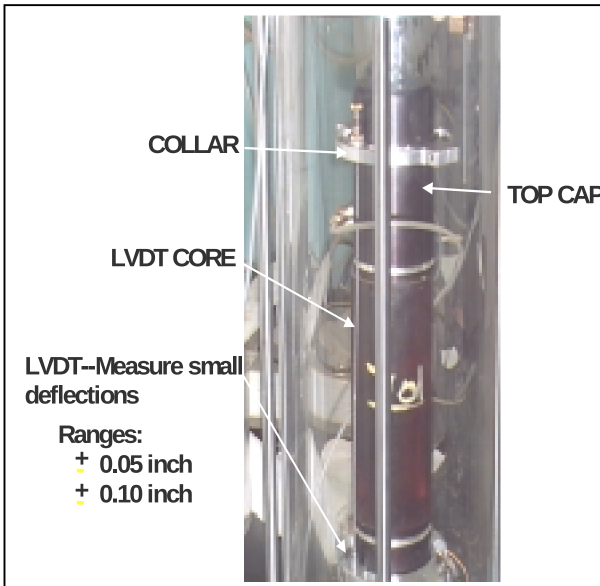

Figure 79. View of LVDTs mounted on the sides of the specimen inside the triaxial chamber.

ANALYSIS

Comparisons of Index Properties of Stabilized and Non-stabilized Soils

Soil Classifications

Initially, about 70 percent of the untreated soils in the subgrades stabilized with hydrated lime were classified, according to the Unified Soil Classification System, as CL, or clays of low to medium plasticity. About 14 % of the soils were classified as CH (fat clays of high plasticity) or MH (silty clay—liquid limit is equal to or greater than 50 %). Hence, about 82 % of the soils were clayey soils. The other soil types (18 %) were classified as SC (silty clay), SM (sandy silt), and ML (silt). About 64 % of the soils were classified as ML, or silt after mixing with hydrated lime,. About 29 % of the soils were classified as SM, or sandy silt. The clayey soils had basically been changed to a silty, or a sandy silty, soil.

The majority of soils in the LKD sections were classified as ML (50 %), SC (38 %), or SM (12 %). After treatment, the soils were either classified as ML (70 %) or SM (30 %). Untreated soils in the cement subgrade sections were classified as CL (37 %), SM (25 %), ML (25%), or SC (7 %). After treatment, the soils were either classified as ML (56 %) or SM (44 %). Untreated soils in the lime/cement-treated section of roadway were classified as CL. After treatment with lime and cement, the stabilized subgrade soils were classified either as ML (83 %), or SM (17 %).

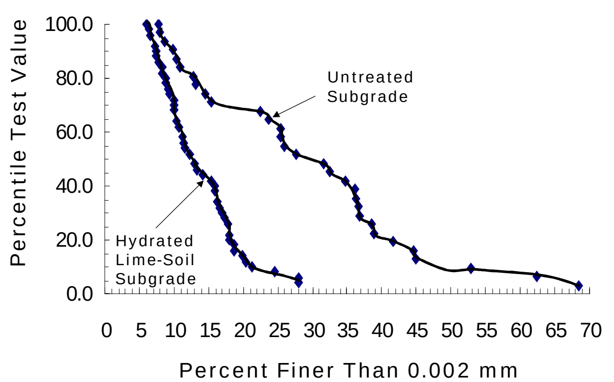

Figure 80. Percentile test value versus the percent finer than 0.002 mm(clay fraction)—hydrated lime.

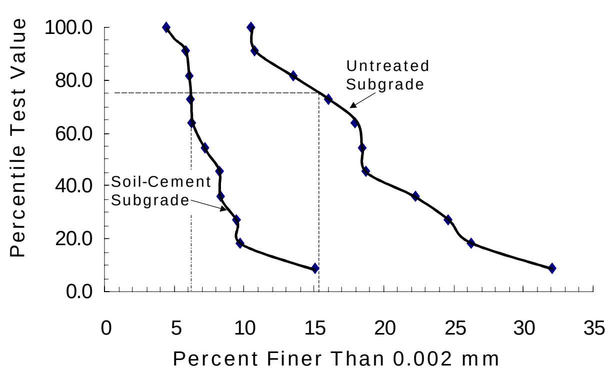

Figure 81. Percentile test versus the percent finer than 0.002 mm (clay fraction)—cement.

Grain sizes—clay fraction

The percentile test value as a function of the clay fraction of the untreated soils and subgrade soils mixed with hydrated lime are compared in Figure 80. Clay fraction is defined here as the percentage of particles in the soil matrix that is finer than the 0.002 mm size. Mixing clayey soils with hydrated lime changes the particle sizes of the subgrade soils. The hydrated lime causes a significant reduction in the clay fraction of the soils. Soils with large values of clay fractions generally have very poor engineering properties and performances. By reducing the clay fraction in soils, the engineering properties, such as shear strength and bearing capacity, improve and increase.

At the 75th percentile test value, the clay fraction of the untreated soil is about 15 %. After mixing with hydrated lime, the clay fraction of the soil is reduced to about 9 %. At the 10th percentile test value, the clay fraction of the untreated soil is about 47 % and after treatment, the clay fraction is reduced to 21 %, as shown in Figure 80. Below the 75th test value, the reduction in clay fraction generally increases from 6 % to 26 %, which represents a significantly reduction.

As shown in Figure 81, mixing soils with Portland cement causes a reduction in clay fraction. At the 75th percentile test value, the clay fraction is about 15 % and, when mixed with Portland cement, the clay fraction decreases to about 6 percent. The percentage of reduction increases as the percentile test value decreases. At the 100th percentile test value, the reduction is 6 %, while at the 10th percentile test value the reduction is about 17 %. Hence, treatment of soils with cement changes the particle sizes of the soil and improves the engineering properties of the materials.

Table 19. In situ CBR values at the 85th percentile test value and structural layer coefficient of treated sections.

Chemical Admixture	In Situ CBR at the 85th Percentile Test Value	Structural Layer Coefficient, a_3
Hydrated Lime	27	0.106
Portland Cement	59	0.127
Hydrated Lime/Portland Cement	32	0.11
Lime Kiln Dust	24	0.10
AFBC	9	0.08
Untreated soil subgrade	2	0.038
Design assumption for untreated subgrade	1.3	0.026

In Situ CBR Values of Untreated and Treated Soil Subgrades

An in situ CBR-percentile test value curve was developed for each chemical admixture, as illustrated in Figure 82. For example, all in situ CBR data obtained at the various sites for each chemical admixture were combined and a curve of the percentile test value as a function of the in situ CBR value was developed. Assuming that the 85th percentile test value is an acceptable design level, a design CBR value for each soil subgrade treated with each chemical admixture was obtained from the curves in Figure 82. Values of CBR, occurring at the 85th percentile test value for each chemical admixture, are summarized in Table 19. In situ test values occurring at the 85th percentile test value for hydrated lime, Portland cement, hydrated lime/cement, and LKD were 27, 59, 32, and 24, respectively. The in situ CBR value at the 85th percentile test value of the untreated subgrade soils was only

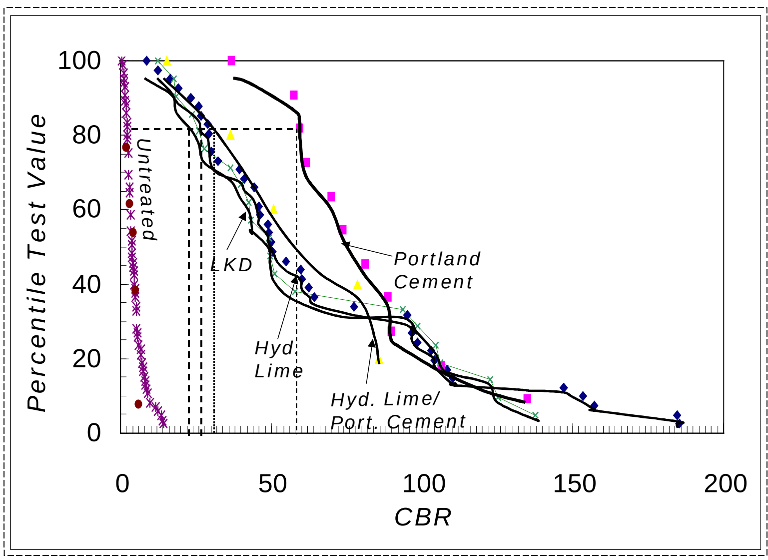

Figure 82. Percentile test value as a function of in situ CBR of stabilized subgrades.

2. The corresponding CBR design assumption of the untreated soil subgrade was only 1.3. In situ CBR values at the 85th percentile test value of the soil subgrades treated with chemical admixtures were approximately 12 to 30 times greater than the in situ CBR of the untreated soil subgrade.

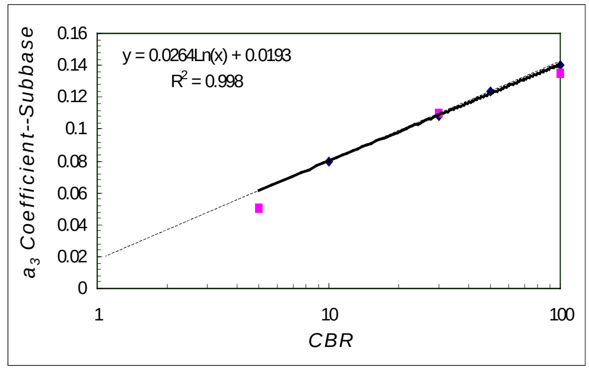

Figure 83. Relationship between the coefficient, a_3, for different subbases at the AASHO Road Test (1960), and CBR.

Determination of the Structural Layer Coefficients, a_3, of Subgrades

A relationship between the structural coefficient, a_3, of subbase and CBR was presented in the 1960 AASHO Road Test. A reproduction of this relationship is shown in Figure 83. The relationship may be expressed as

$$a_3 = 0.0264\ln(CBR) + 0.0193. \quad (1)$$

Inserting values of CBR occurring at the 85th percentile, as given in Table 19, design values of a_3 may be estimated from Equation 2 for the subgrades treated with chemical admixtures. In this case, the chemically treated layer is considered a subbase material.

Moisture Contents of Non-Stabilized and Stabilized Soil Subgrades

Non-Stabilized Subgrades

During field operations, moisture contents were obtained at all locations where in situ CBR tests were performed. Samples obtained from those borings were located at the top of the subgrade, as depicted in Figure 84. Moisture contents of the subgrades were also obtained at depths below the top of the in situ CBR testing positions, at the top of the treated subgrades, and at depths below the CBR locations. Moisture content data are tabulated in Appendix B, Tables B-1 through B-13.

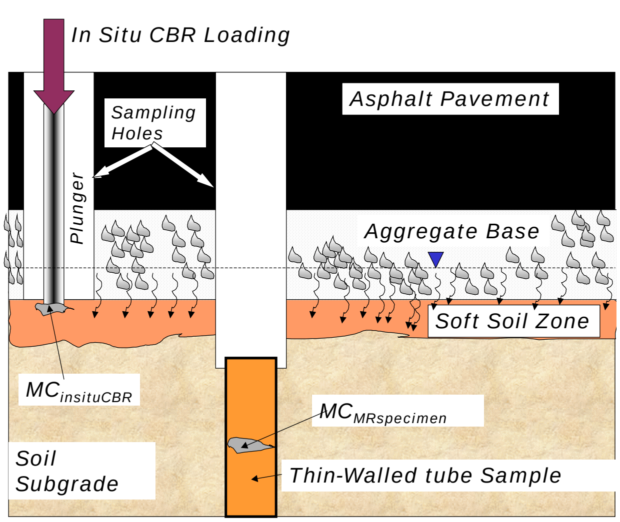

Figure 84. Soft zone of soil situated at the top of the soil subgrade (untreated section).

Subgrade moisture contents measured in previous research (Hopkins and Beckham, 1993), indicated that, oftentimes, a thin soft zone of soil exists in the top portion of untreated soil subgrades, as depicted in Figure 84. To test this observation, moisture contents measured at the tops of the untreated subgrades where in situ CBR tests were performed, and moisture contents at depths below the tops of the untreated subgrades were compared, as shown in Figure 85. The moisture contents were graphed and

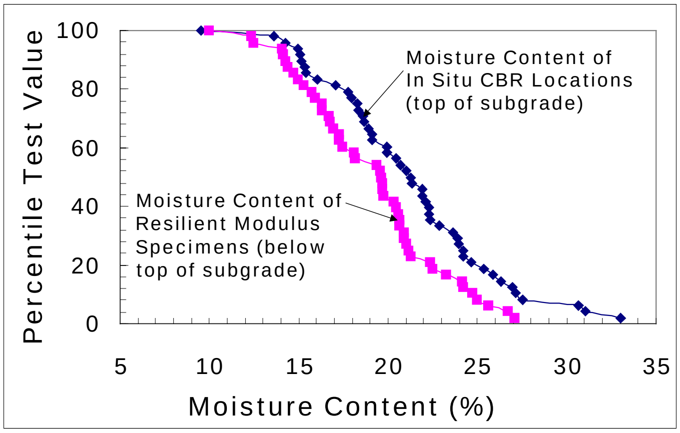

Figure 85. Comparison of moisture contents of specimens obtained from in situ CBR locations and resilient modulus specimens obtained below the top of the untreated soil subgrades.

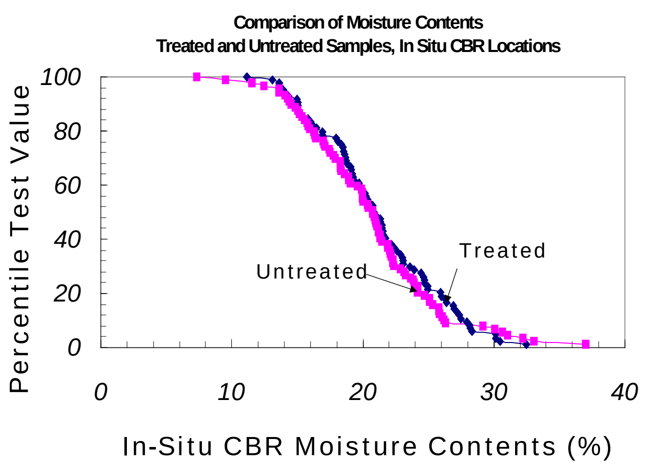

Figure 86. Comparison of moisture contents of specimens obtained from in situ CBR locations at the tops of the stabilized layer and non stabilized layer.

compared as a function of the percentile test value. At the 80th and 15th percentile test values, the moisture contents at the in situ CBR locations (tops of untreated subgrades) were about 3 percent more then the moisture contents of samples obtained from locations below the tops of the untreated subgrades. As shown by previous research (Hopkins and Beckham, 1993), in situ and laboratory CBR values of Kentucky clayey soils, when first compacted, range from about 10 to 40. However, after soaking, CBR values decrease to values ranging from about 0.5 to 6. As shown in Figure 82, the in situ CBR values at the tops of the untreated soil subgrades ranged from about 1 to 6.

Stabilized Subgrades

At the in situ CBR locations, moisture contents obtained at the very top of the chemically stabilized subgrades were compared to moisture contents at the tops of the non-stabilized subgrades. The relationships, in the form of percentile test values as function of the two different sets of moisture contents, are shown in Figure 86. Although in situ moisture contents at the tops of the treated subgrades are nearly identical to the in situ moisture contents of the tops of the non-stabilized subgrades, large discrepancies exist between in situ CBR values at the tops of the stabilized subgrades and the in situ CBR values at the tops of the non-stabilized subgrades, as shown in Figure 82 and Table 19. At the 85th percentile test value, the in situ CBR values of the stabilized subgrades (excluding the AFBC admixture) are some 12 to 30 times larger than CBR values of the non-stabilized subgrades. Apparently, the chemical admixtures "lock-up" the moisture content chemically so that it does not affect the strength of the stabilized materials.

A comparison is shown in Figure 87 of all moisture contents at all locations in the treated and untreated subgrades. Moisture contents measured at the tops (in situ CBR locations) of the treated and untreated subgrades and at locations below the tops of the treated subgrades are nearly identical. However, moisture contents measured at locations below the tops of the untreated subgrade are lower (about 3 percent) than the moisture contents measured at the tops (in situ CBR locations) of the treated and untreated subgrades and at locations below the tops of the treated subgrades. The fact that the moisture contents at the top of the treated subgrades, and at locations below the tops of the treated subgrades, are nearly equal shows that the moisture content is fairly uniform throughout the treated layer. Moreover, the fact that the moisture contents at the tops of the treated subgrades are greater than the moisture contents below the tops of the untreated subgrades is not unexpected. Based on previous experiences (Hopkins and Beckham, 1993), when clayey soils are mixed with chemical admixtures and compacted, the optimum moisture content of compacted soil-chemical admixture is greater than the optimum moisture of the compacted soil without a chemical admixture.

Back Calculation of Structural Layer Coefficient, a_3

Using the 1993 AASHTO Design Guide

In the early development of Kentucky's chemical stabilization program during the mid-eighties, structural credit of the treated soil subgrades was usually not assumed in the pavement design. Soil subgrades treated with chemical admixtures were merely treated as "working platforms" and not considered a structural member of the pavement. As one approach of confirming, or checking, that the treated soil subgrades of the selected study sections were not considered as part of the pavement structure in the original design analysis, back calculations were performed to estimate the layer coefficient, a_3, using the expression from the AASHO Test Road (1960), or

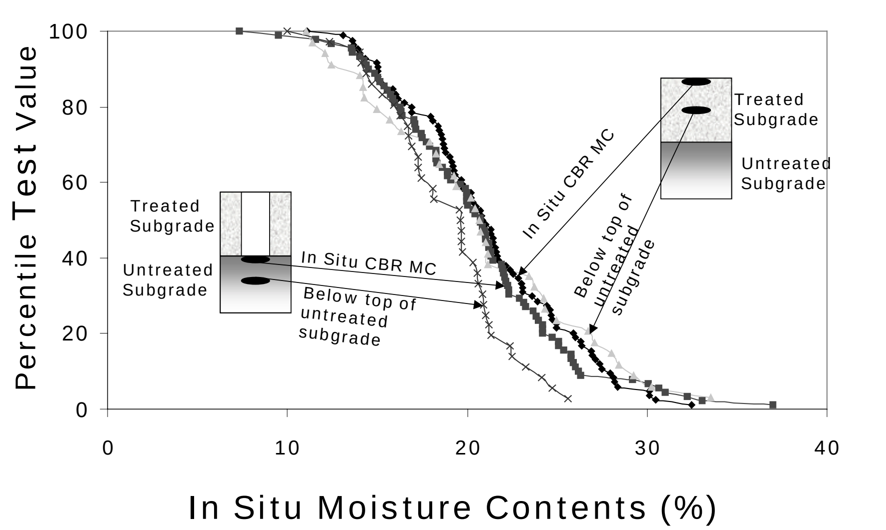

Figure 87. Comparison of moisture contents of specimens obtained at all locations in the treated and untreated subgrades.

$$SN = a_1D_1 + a_2D_2 + a_3D_{3+.........}a_iD_i\,, \tag{2}$$

where

SN = structural number,

a_1, a_2, and a_3 = layer coefficients representative of surface, base, and subbase (in this case, the treated layer), respectively, and

d_1, d_2, and d_3 = actual thickness, in inches, of surface, base, and subbase courses (in this case, the chemically treated, soil subgrade), respectively.

Back-calculations were made using the AASHTO design equation, which appears in the 1993 Design Guide. This equation is as follows:

$$Log(D_D D_L W_{18}) = Z_R S_0 + 9.36 Log_{10}(SN+1) - 0.20 + Log_{10}\left[\frac{\frac{\Delta PSI}{4.2-1.5}}{0.40+\frac{1094}{(SN+1)^{5.19}}}\right] + 2.32 Log_{10} M_R - 8.07 \qquad (3)$$

where

D_D = a directional distribution factor, expressed as ratio, that accounts for the Distribution of ESAL units by direction, e.g., east-west, north-south, etc.,
D_L = a lane distribution factor, expressed as a ratio, that accounts for distribution of traffic when two or more lanes are available in one direction,
W_{18} = the cumulative two-directional 18-kip ESAL units predicted for a specific section of highway during the analysis period (from planning group),
Z_R = values obtained from standard normal curve area tables.
SN = (defined by Equation 2),
ΔPSI = p_0-p_t = design serviceability loss
p_t = initial serviceability index (from AASHO Road Test, p_t = 4.2),
p_0 = terminal serviceability index (from AASHO Road Test, p_t = 2.5 or 2.0),
M_r = resilient modulus of soil subgrade.

To facilitate back-calculations, Equation 3 was programmed for the computer in a client-server environment. A Graphical User Interface for data entry and Equation 3 were scripted using software referred to as PowerBuilder® 7.0. To facilitate the use of this program, and to make it readily available to many users, the computer program was entered into the "Applications" Section" of the Kentucky Geotechnical Database, which resides on a server in Frankfort Kentucky (Hopkins et al, 2002). The data entry, graphical user interface is illustrated in Figure 88. Several offices of the Kentucky Transportation Cabinet, including the Geotechnology Section of the University of Kentucky Transportation Center are connected to this database and server. In performing the back-calculations, values of the layer coefficients, a_1 and a_2, appearing in Equations 2 and 3, and which correspond to the asphalt and base aggregate layers, were assumed to be 0.44 and 0.14, respectively. These values were obtained from the 1993 AASHTO Guide. Design personnel of the Kentucky Transportation Cabinet provided design values of ESAL and CBR for each pavement section. Values of a_3

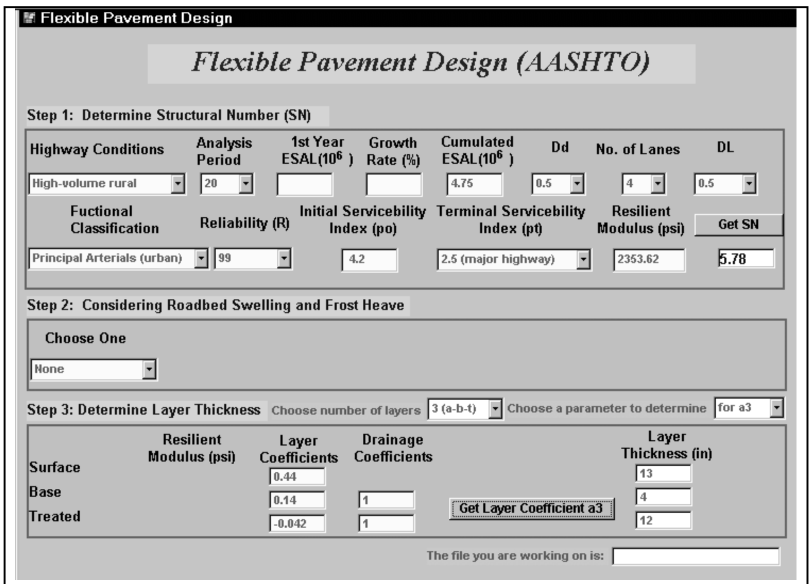

Figure 88. Graphical User Interface (data entry screen) of a computer program for performing back-calculations to determine the layer coefficient, a_3, of the chemically-treated soil subgrade (subbase).

used in the back calculations of the chemically stabilized layers are shown in Table 20. The stabilized layers were assumed to be a subbase in the calculations.

Based on the design values of ESAL and CBR, a design structural number, SN_R, was computed for each roadway section using the AASHTO procedure (Figure 88). A value of resilient modulus was assumed based on the relationship

$$Mr = 1500CBR\ , \tag{4}$$

which is frequently cited in the literature (Hopkins et al, 1993). The actual structural number, SN_a, based on typical measurements of the pavement layers at each roadway section was computed. A back-calculated structural layer coefficient for each section was computed based on the following expression

$$a_3 = \frac{\Delta SN}{t_{stab}} = \frac{SN_R - SN_a}{t_{stab}}\ , \tag{5}$$

where t_{stab} is the thickness of the stabilized layer at a given site. Structural numbers, SN_R and SN_a, obtained from the calculations and back-calculations of a_3 of each roadway section using the AASHTO design framework are shown in Table 20. Except for six sections of the Lee County site (KY 11), the McCreary County site, and the Shelby county site, the back-calculated values of the sections were near zero, or slightly less than zero. In those cases, no structural credit was given to the stabilized layers, since the a_3 coefficients of the stabilized layers were near zero. The back calculated values of the structural coefficient, a_3, of six sections of Lee-Wolfe Counties, McCreary, and Shelby County ranged from values of 0.08 to 0.11. The coefficient, $a_{3,}$ of the two sections (numbers 2 and 5) of roadways in Lee and Wolfe Counties were 0.10 and 0.09, respectively. Sections 3 and 4, which contained the soil-hydrated lime and soil-LKD subgrades had an a_3-coefficient of 0.03 to 0.01 and 0.04, respectively—essentially no structural credit was given to this stabilized section. Similarly, a_3 of the untreated section (number 6) had a value of –0.02. The two sections (numbers 1 and 7 had values of 0.03 and 0.09, respectively. At the Shelby County, the coefficient ranged from 0.08 to 0.11. Hence, in this case, structural credit was given to this section of roadway. In cases where the subgrades had positive coefficients, structural credit had been given to the stabilized subgrades and in each case no overlays have been used. In those cases, the pavements have been in place from 8 to 15 years.

Using the Kentucky Design Curves

Because values of ESAL in the Kentucky Design Procedure are computed in a different fashion than values of ESAL in the AASHTO Design Guide, back-calculations of a_3 were performed using the Kentucky Design Curves, as formulated by Southgate (1981). Using design values of ESAL computed by the Kentucky Design Procedure in the AASHTO Design Procedure could be viewed to be improper. Consequently, the Kentucky design procedure was programmed for the computer so that back-calculations of a_3 could be performed. The original data used to construct the design curves was obtained from Southgate in 2000. Using a finite difference technique, the Kentucky Design curves were programmed for the computer and a “windows” software program

Table 20. Summary of back-calculated values of the layer coefficient, a_3, of treated subgrades

Site	Subgrade Type	Design CBR[3]	Design ESAL[4] (mil.)	Measured Average Pavement Thickness (Inches) Asph.	Base	Subgrade	Back-calculated structural layer coefficient, a_3[5], using AASHTO Equation 3
Anderson	H. Lime	2	3.2	13	4	12	-0.06
Boyle	H. Lime	4	9.2	14	4	8-12	-0.12
Fayette	H. Lime	3	4.75	12	8	10	-0.11
Lee/Wolfe							
Section 1	AFBC[1]	2	1.3	9	5	12	0.03
Section 2	P. Cement	2	1.3	7	5	12	0.10
Section 3	H. Lime	2	1.3	9-9.5	5	12	0.02
Section 4	LKD[2]	2	1.3	9	4	12	0.04
Section 5	PC	2	1.3	7.5	5	12	0.09
Section 6	Untreated	2	1.3	10	6	0	-0.02
Section 7	AFBC	2	1.3	7.5	5-6	12-14	0.07
McCreary	P. Cement	2	1.3	8	4	12	0.04
Shelby	H. Lime	2	2.4	10.5-11	0.0	8	0.07
Hardin	H. Lime	3	1.5	11	4	8(16)	-0.08
		3	4.4	11	4	8(16)	-0.02
Owen	H. Lime	2	0.6	10	4	8	-0.04
Trigg	H. Lime	6	3.4	10	6	12	-0.11
Boone	H. Lime	5	0.65	9	4-6	8-14	-0.13
McCracken	LKD	11	2.0	8	4	12(Bank) & 12 LKD	-0.05
Hickman	LKD	5	1.3	9	6	13-20	-0.07
Breckinridge	P. Cement	4	1.9	10	5	16	-0.06
Daviess	P. Cement	6	1.0	8	0.0	20-24	0.01

1. *A byproduct, Atmospheric Fluidized Bed Combustion ash, from a Kentucky oil refinery*
2. *A byproduct resulting from the production of hydrated lime.*
3. *Value of CBR assumed in the design of pavement section.*
4. *Value of ESAL per lane assumed in design.*
5. *From AASHTO Guide for design of Pavement Structures (1993).*

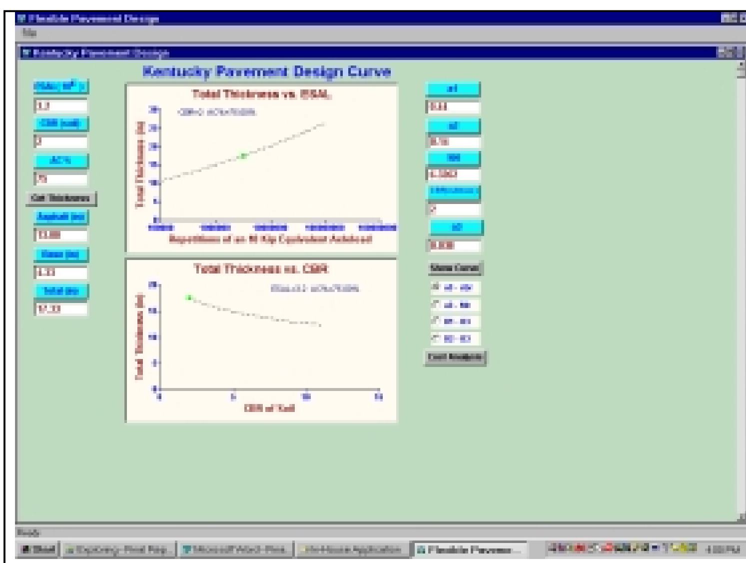

Figure 89. Graphical User Interface for obtaining thickness of pavement components and structural number from the Kentucky Design Curves

was developed. The data entry graphical user interface for this program is shown in Figure 89. By entering the design values of ESAL and CBR value, thickness of asphalt concrete and the aggregate base are obtained. The program was also designed to compute the structural Number, SN_R, based on the thickness of the asphalt layer and aggregate base. Values of a_1 and a_2, respectively, were assumed to be 0.44 and 0.14. By entering the design CBR, a back-calculated value of a_3 (subbase) may be obtained. A summary of the back-calculated values of the coefficient, a_3, are

Table 21. Summary of back-calculated values of the layer coefficient, a_3, of treated subgrades

Site	Subgrade Type	Design CBR[3]	Design ESAL[4] (10^6)	Design SN_R[5]	Measured Average Pavement Thickness (Inches) Asphalt	Base	Subgrade	SN_a[6] Actual (w/o Stabilized Subgrade)	ΔSN[7]	Back-calculated structural coefficient, a_3[8], using Ky Curves
Anderson	H. Lime	2	3.2	6.32	13	4	12	6.28	0.04	0.00
Boyle	H. Lime	4	9.2	6.55	14	4	8-12	6.72	-0.17	-0.01
Fayette	H. Lime	3	4.75	6.44	12	8	10	6.40	0.04	0.00
Lee/Wolfe										
Section 1	AFBC[1]	2	1.3	5.73	9	5	12	4.66	1.07	0.09
Section 2	P. Cement	2	1.3	5.90	7	5	12	3.78	2.12	0.18
Section 3	H. Lime	2	1.3	5.83	9-9.5	5	12	4.77	1.06	0.09
Section 4	MKD[2]	2	1.3	5.74	9	4	12	4.52	1.22	0.10
Section 5	P Cement	2	1.3	5.88	7.5	5	12	4.00	1.88	0.16
Section 6	Untreated	2	1.3	5.85	10	6	0.0	5.24	0.61	0.05
Section 7	AFBC	2	1.3	5.90	7.5	5-6	12-14	4.07	1.83	0.15
McCreary	P. Cement	2	1.3	6.67	8	4	12	4.08	1.16	0.10
Shelby	H. Lime	2	2.4	5.96	10.5-11	0.0	8	4.73	1.23	0.19
Hardin	H. Lime	3 3	1.5 4.4	5.37 6.25	11 11	4 4	8(16) 8(16)	5.4 5.4	-0.07 0.85	0.00 0.05 to 0.10
Owen	H. Lime	2	0.6	5.16	10	4	8	4.96	0.2	0.03
Trigg	H. Lime	6	3.4	5.35	10	6	12	5.24	0.11	0.00
Boone	H. Lime	5	0.65	4.39	9	4-6	8-14	4.66	-0.27	-0.03 to -0.02
McCracken	MKD	11	2.0	4.21	8	4	12(Bank) & 12 MKD	5.04	4.08	0.01
Hickman	MKD	5	1.3	4.85	9	6	13-20	4.8	0.05	0.00
Breckinridge	P. Cement	4	1.9	5.32	10	5	16	5.10	0.22	0.01
Daviess	P. Cement	6	1.0	4.35	8	0.0	20-24	3.52	0.83	0.04

1. A byproduct, Atmospheric Fluidized Bed Combustion ash, from a Kentucky oil refinery 2. A byproduct resulting from the production of hydrated lime. 3. Value of CBR assumed in the design of pavement section. 4. Value of ESAL per lane assumed in design. 5. Based on the design values of CBR and ESAL, the value of SN obtained from the Kentucky Design Curves. 6. The actual SN value based on the measured thickness of the different pavement components. 7. $\Delta SN = SN_R - SN_a$ 8. $A_3 = \Delta SN$/Thickness of stabilized layer.

shown in Table 21 for each roadway section. The stabilized subgrade was assumed to perform as a subbase.

At 11 study sections, no structural credit was given to the chemically stabilized subgrade. The a_3 coefficients of the subbases of the Anderson, Boyle, Fayette, Hardin, Owen, Trigg, Boone, McCracken, Hickman, Breckinridge, and Daviess County sites ranged from –0.03 to 0.05. Essentially, those roadway subgrades were assigned no structural credit, although they had been stabilized. At one part of the Hardin County site, the layer coefficient ranged from 0.05 to 0.10—some credit was given--while in another part the coefficient was 0.00. The subgrade of Section 6 of the Lee-Wofle County site was not stabilized. The coefficient of that subgrade was 0.05. At eight sites, structural credit had been given to the treated subgrades and the "in service" layer coefficients ranged from 0.09 to 0.19, Pavements at those sites have performed very well. Based on the good performances of pavements where the in-service layer coefficients, a_3 (back calculated) ranged from 0.09 to 0.19, the design layer coefficients proposed in Tables 19 and 22 appear very reasonable.

Table 22. Summary of "in service" a_3 coefficients.

Chemical admixture	In Situ CBR at the 85th Percentile Test Value	Structural Layer coefficient[1], a_3	Roadway Site	In-Service Coefficient[2], a_3
Hydrated Lime	27	0.106	Hardin County Section 3, KY 11 Shelby County	0.05-0.10 0.09 0.19
Portland Cement	59	0.127	Section 2, KY 11 Section 5, KY 11 McCreary County	0.18 0.16 0.10
Hydrated Lime/Portland Cement	32	0.11	--	--
LKD	24	0.10	Section 4, KY11	0.10
AFBC	9	0.08	Section 1, KY 11 Section 7, KY 11	0.09 0.15
Untreated soil subgrade	2	0.038	Section 6, KY 11	0.05
Design assumption for untreated subgrade	1.3	0.026	--	--

1. *Based on the in situ CBR at the 85th percentile test value and the curve given in Figure 83.*
2. *Back-calculated value using the Kentucky Design Curves (Southgate et al 1981)*

Resilient Modulus of Undisturbed Core Specimens from Stabilized and Non-Stabilized Subgrades

Mathematically, resilient modulus, M_r, has been defined as:

$$M_r = \frac{\sigma_d}{\varepsilon_a} \quad , \tag{6}$$

where

σ_d = deviator stress = $\sigma_1 - \sigma_3$,
σ_1 = major principal stress,
σ_3 = minor principal stress, and
ε_a = axial strain recoverable after the release of the deviator stress.

Deformation properties of soils are not constant. They are determined by both intrinsic properties of soils and the stresses applied to the soils. A number of mathematical models have been proposed for modeling the resilient modulus of soils and aggregates. Most mathematical expressions relate resilient modulus, the dependent variable, to one independent variable, either the deviator stress, σ_d, or confining stress, σ_3, or the sum of principle stresses, σ_{sum} ($\sigma_1 + \sigma_2 + \sigma_3$), or to two independent variables, σ_d and σ_3. Some widely published resilient modulus models are examined below. As shown by this review and analysis of available models, only two models are used in the analyses of resilient modulus data reported herein.

Review of Mathematical Models for Relating Resilient Modulus and Stresses

Moossazadeh and Witczak (1981) proposed the following relationship for presenting resilient modulus data (Model 1):

$$M_r = k_1 \left(\frac{\sigma_d}{p_a} \right)^{k_2}, \tag{7}$$

where k_1 (y-intercept) and k_2 (slope of the line) are coefficients obtained from a linear regression analysis and p_a is a reference pressure. In this model, the effect of the confining stress is not considered.

Dunlap (1963) suggests the following relationship (Model 2):

$$M_r = k_1 \left(\frac{\sigma_3}{p_a} \right)^{k_2}, \tag{8}$$

where k_1 and k_2 are regression coefficients and σ_3 is the confining stress. The influence of the deviator stress is ignored in this relationship.

Seed et al (1967) suggests that the resilient modulus is a function of the sum of the principle stresses, or (Model 3)

$$M_r = k_1 \left[\frac{\sigma_{sum}}{p_a} \right]^{k_2}, \tag{9}$$

where σ_{sum} is the sum of principal stresses ($\sigma_1 + \sigma_2 + \sigma_3$) , or for the triaxial compression test, $\sigma_1 + 2\sigma_3$). This expression appears in the AASHTO Pavement Design Guide (1993) and in the testing standard, AASHTO T 292-91(2000). This relationship does not account for the effect of confining stress on the resilient modulus. Relationships given by Equations 8 and 9 do not consider the effect of shear stress on the resilient modulus of soils.

May and Witczah (1981) and Uzan (1985) propose another model that considers the effects of shear stress and the confining stress and deviator stress, or (Model 4)

$$M_r = k_1 \left(\frac{\sigma_{sum}}{p_a} \right)^{k_2} \left(\frac{\sigma_d}{p_a} \right)^{k_3}, \tag{10}$$

where k_1, k_2, and k_3 are correlation regression coefficients. Under identical loading ($\sigma_1 = \sigma_2 = \sigma_3$), Uzan's model will lead to a value of M_r that either goes to zero when the coefficient, $k_3 > 0$, or, M_r will become infinite in the case of $k_3 < 0$. In all of the models cited above, a regression fit can be made for a selected confining stress. However, when the confining stress changes, the coefficients change.

To correctly model the resilient modulus of soils and aggregates and to account for the influences of confinement stress and deviator stress, a new model (Hopkins et al, 2001: Ni et al 2001) is proposed, or (Model 5)

$$M_r = k_1\left(\frac{\sigma_3}{p_a}+1\right)^{k_2}\left(\frac{\sigma_d}{p_a}+1\right)^{k_3}. \qquad (11)$$

In this model, the coefficients, k_1 and k_2, will always be positive. For most situations the coefficient, k_3, is negative for soils and aggregates. As shown by the relationship given by Equation 11, the resilient modulus increases as the confining stress increases. The modulus will increase or decrease, as in most cases, with the increase of shear stress. When both σ_3 and σ_d approach zero, the value of resilient modulus, M_r, approaches the value of k_1, which is the initial resilient modulus value and a property of the soil. How the resilient modulus of soils changes from its initial value depends on the stress path and the stress state applied to the soil mass. The coefficients, k_1, k_2, and k_3, are derived from test data using multiple correlation regression analysis.

Equations 10 and 11 (Models 4 and 5) are based on the assumption that the normal stresses, σ_2 and $\sigma_{3,}$ are equal. If σ_2 is not equal to $\sigma_{3,}$ then Equations 10 and 11 may be written for the more general case, or

$$M_r = k_1\left(\frac{\sigma_{sum}}{p_a}\right)^{k_2}\left(\frac{\tau_{oct}}{p_a}\right)^{k_3} \quad \text{(Model 4)}, \qquad (12)$$

and

$$M_r = k_1\left(\frac{\sigma_3}{p_a}+1\right)^{k_2}\left(\frac{\tau_{oct}}{p_a}+1\right)^{k_3} \text{(Model 5)}, \qquad (13)$$

where

$$\tau_{oct} = \frac{\sqrt{2}}{2}(\sqrt{(\sigma_1-\sigma_2)^2+(\sigma_2-\sigma_3)^2+(\sigma_3-\sigma_1)^2} \qquad (14)$$

and,

$$\tau_{oct} - Octahedral \quad shear \quad stress \quad acting \quad on \quad the \quad material.$$

When σ_2 is equal to σ_3 (the triaxial case), Equation 14 reduces to:

$$\sigma_1 - \sigma_3 = \sigma_d = deviator \quad stress.$$

Consequently, Equations 12 and 13 become Equations 10 and 11.

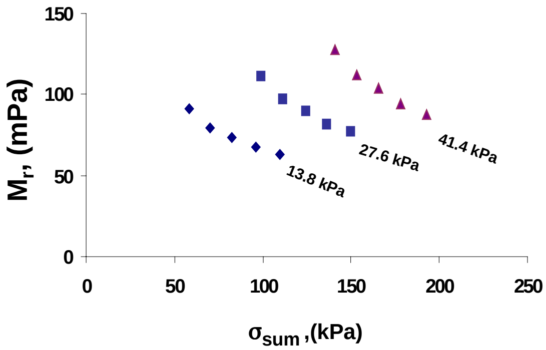

Figure 90. Resilient modulus, M_r, as a function of the sum of the principal stresses, σ_{sum}.

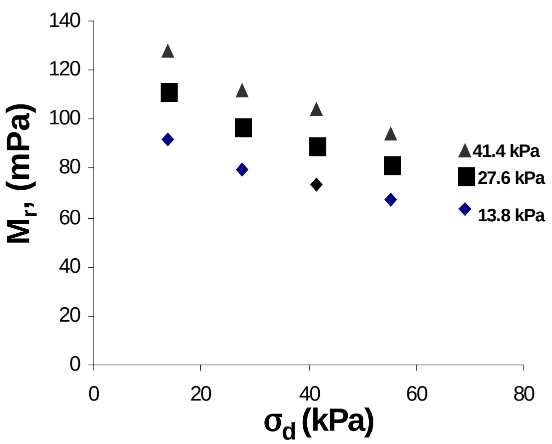

Figure 91. Relationship between deviator stress, σ_d, and resilient modulus, M_r.

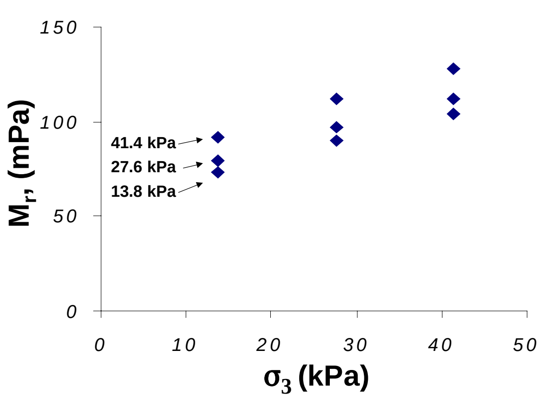

Figure 92. Relationship between confining stress, σ_3, and resilient modulus, M_r.

Comparisons of Resilient Modulus Models

Simple correlation analysis--- To evaluate the different models cited above, 72 laboratory specimens of different types of soils were compacted and resilient modulus tests were performed. Specimens used in this series of tests were compacted to 95 % of maximum dry density and optimum moisture (ASSHTO T-99). Resilient modulus data generated from those tests have been published elsewhere (Hopkins et al and Ni et al, 2002). Resilient modulus data shown in Figures 90 through 92 are typical of the type of data obtained from the resilient modulus tests. In Figure 90, the relationship between resilient modulus and the sum of the principal stresses is shown. Three data sets shown in this figure correspond to confining stresses of 13.8, 27.6, and 41.4 kPa (2, 4, and 6 psi, respectively). The relationship between deviator stress and resilient modulus is shown in Figure 91 and the three data sets correspond to confining stresses of 13.8, 27.6, and 41.4 kPa (2, 4, and 6 psi). Similarly, in Figure 92, the relationship between confining stress and resilient modulus is shown. The three data sets correspond to confining stresses of 13.8, 27.6, and 41.4 kPa. The data curves depicted in Figures 90 through 92 illustrate that confining and deviator stresses have different effects on the resilient modulus of soils. Under a constant confining stress, the resilient modulus of soils decreases as the deviator stress increases, as shown in Figure 91. If the deviator stress is held constant, then the resilient modulus increases as the confining stress increases.

Model 1 ($M_r = k_1(\sigma_d / p_a)^{k_2}$) does not consider the effect of the confining stress on resilient modulus of soils while Model 2 ($M_r = k_1(\sigma_3 / p_a)^{k_2}$) does not consider the effect of deviator stress on resilient modulus. Therefore, these two models have limited use. Although Model 3 ($M_r = k_1(\sigma_{sum} / p_a)^{k_2}$) includes the sum of principle stresses, and $\sigma_{sum} = \sigma_1 + \sigma_2 + \sigma_3 = 3\sigma_3 + \sigma_d$, the model only contains one independent variable, σ_{sum}. The effects of both confining stress and deviator stress of this model are not considered as independent variables. Although

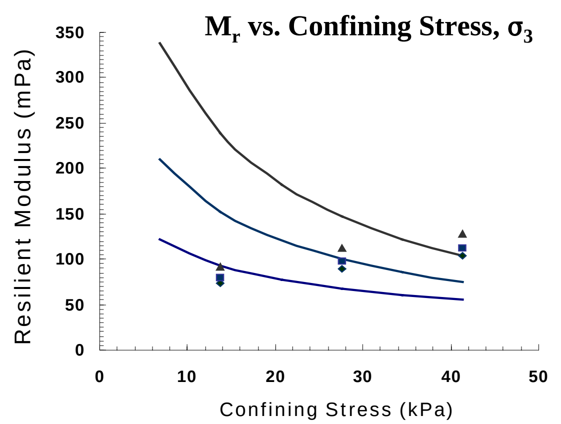

Figure 93. Prediction of relationship between resilient modulus, M_r, and confining stress, σ_3, using Model 3.

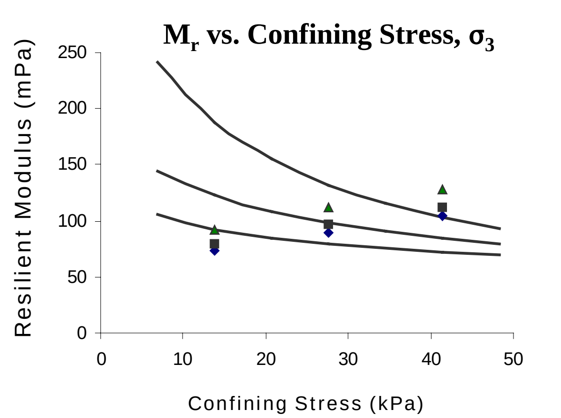

Figure 94. Prediction of relationship between resilient modulus, M_r, and confining stress, σ_3, using Model 4.

Model 4 ($M_r = k_1(\sigma_{sum} / p_a)^{k_2}(\sigma_d / p_a)^{k_3}$) does consider the effects of both the sum of the principle stresses and deviator stress on the resilient modulus, the coefficients k_1, k_2, and k_3 vary significantly when simple regression analysis is performed for each confining stress. However, as shown below, when multiple regression analysis is performed on all data points the relationship for Model 4 improves.

Resilient modulus test data indicate that as the deviator stress increases the resilient modulus decreases, but as the confining stress increases, the resilient modulus tends to increase. Any one of the three data sets in Figure 91 could be used to obtain the correlation coefficients, k_1 and k_2, from a simple regression analysis. If Model 3 correctly represents the relationship between resilient modulus and stress state, then the values of k_1 and k_2 should be nearly the same for each curve. As shown in Table 23, the value of k_1 ranges from 305,213 to 4,739,146 while k_2 varies from –0.572 to –1.202. Figure 93 shows the results of using Model 3 to predict the relationship between resilient modulus and confining stress using the three sets of k_1 and k_2 values obtained from the simple regression analysis. Model 3 does not correctly include the effects of confining stress on resilient modulus. In Figure 94, regression results from Model 4 are shown. The three sets of correlation coefficients, k_1, k_2, and k_3, obtained from regression analysis are shown in Table 23. The correlation coefficients (k_1, k_2, and k_3) of Model 4 vary significantly.

To model the relationship between resilient modulus of soils (and aggregates) and stress state correctly, the following model (Equation 11, or 13) has been proposed:

$$M_r = k_1\left(\frac{\sigma_3}{p_a}+1\right)^{k_2}\left(\frac{\sigma_d}{p_a}+1\right)^{k_3}.$$

This model considers separately the effects of deviator stress and confining stress on the resilient modulus. When σ_3 and σ_d approach zero, M_r approaches the coefficient k_1. Therefore, k_1 is the initial resilient modulus of the soil before any load is applied. Test data appearing in Figures 91 and 92 are used in a simple regression analysis to obtain the coefficients, k_1, k_2, and k_3, of the new model. Results are shown in Table 23. Although the confining stress changes, the value of the each coefficient, k_1, k_2, or k_3, is nearly the same. For instance the three different values of the coefficient, k_1, range only from 80,479 to 80,844, or a difference of less than 1 percent.

Table 23. Correlation coefficients of Models 3, 4, and 5.

Confining Stress, σ_3 (kPa)	Model 3 $M_r = k_1\left(\frac{\sigma_{sum}}{p_a}\right)^{k_2}$		Model 4 $M_r = k_1\left(\frac{\sigma_{sum}}{p_a}\right)^{k_2}\left(\frac{\sigma_d}{p_a}\right)^{k_3}$			Model 5 $M_r = k_1\left(\frac{\sigma_3}{p_a}+1\right)^{k2}\left(\frac{\sigma_d}{p_a}+1\right)^{k_3}$		
	k_1	k_2	k_1	k_2	k_3	k_1	k_2	k_3
13.8	305,213	-0.572	176,657	-0.121	-0.270	80,844	0.392	-0.281
27.6	1,209,923	-0.899	419,437	-0.112	-0.467	80,479	0.404	-0.284
41.4	4,739,146	-1.202	1,834,656	-0.066	-0.869	80,765	0.415	-0.286

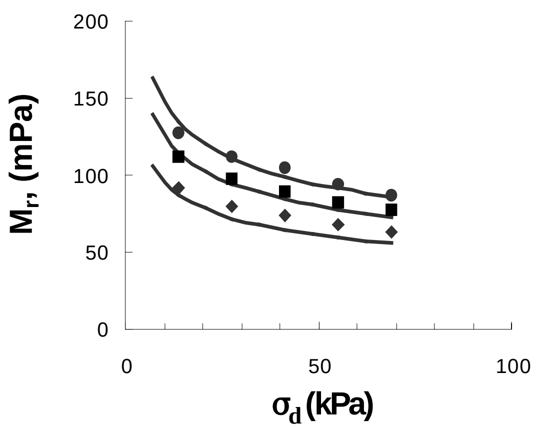

Figure 95. Prediction of the relationship between resilient modulus, M_r, and deviator stress, σ_d, from the new model 5.

Values of the coefficients, k_2 and k_3, range only from 0.392 to 0.415 and –0.281 to –0.286, or a difference of about 5 and 1.7 percent, respectively. As shown in Table 23, any set of constants could be used to predict the relationships between resilient modulus of soils and stress state. For example, the values, k_1 = 80,844, k_2 = 0.392, and k_3 = -0.281, from Table 23 are used in the proposed Model 5 to predict the relationships of the resilient modulus to confining stress, deviator stress, and the sum of the principal stresses. The predicted relationships are compared to the actual test data in Figures 95, 96, and 97, respectively. The results show that the new model predicts the various relationships very well. Moreover, the results also prove that the new model correctly includes the effects of both confining stress and

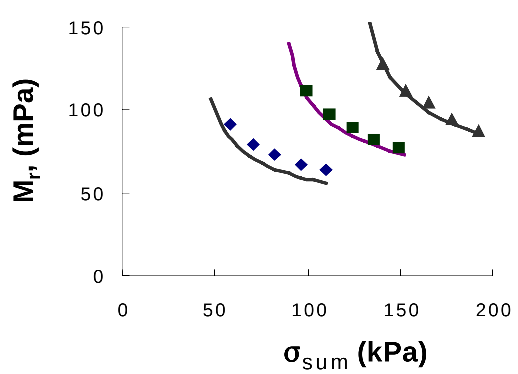

Figure 96. Prediction of the relationship between resilient modulus, M_r, and the sum of the principal stresses, σ_{sum}, from the new model 5.

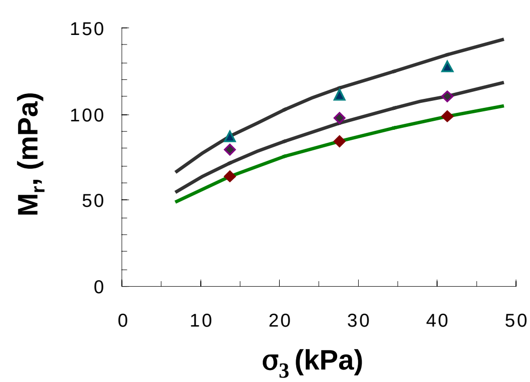

Figure 97. Prediction of the relationship between resilient modulus, M_r, and confining stress, σ_3, from the new model 5.

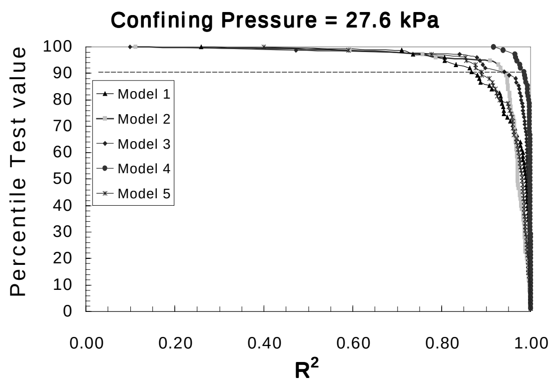

Figure 98. Percentile test value as a function of R^2 obtained for models 1 through 5.

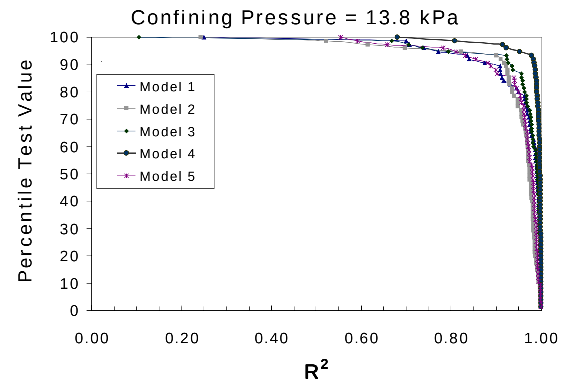

Figure 99. Percentile test value as a function of R^2 obtained for models 1 through 5.

deviator stress on the resilient modulus of soils. Each of the five models provide a reasonable correlation when the confining stress is held constant in the simple correlation analysis, as illustrated in Figures 98, 99, and 100. In each of those Figures, the percentile test value is shown as a function of R^2 for confining stresses of 13.8, 27.6, and 41.4 kPa (2, 4, and 6 psi), respectively. Values of R^2 at the 90th percentile test value are summarized in Table 24. Generally, the value of R^2 was equal to or exceeded 0.90. Although Models 3 and 4 yielded slightly better regression curves than Model 5 for a constant confining pressure, there was much greater variation in the coefficients when all confining curves were considered than the coefficients for Model 5, as illustrated in Table 24. Models 1 and 2 can only be used to determine a regression curve for a constant confining stress or deviator stress. Hence, these two models cannot be used in a general sense and their uses are limited.

Multiple correlation analysis--In the relationships expressed by Equations 7, 8, and 9 (Models 1, 2, and 3), respectively, only two variables are involved. The resilient modulus is a dependent variable while either the deviator stress, confining stress, or sum of principle stresses is an independent variable. Consequently, only simple correlation analysis can be performed on those equations.

However, Models 4 and 5, expressed by Equations 10 and 11, respectively, involve 3 variables. The resilient modulus is the dependent variable and the sum of the principle stresses and deviator stress are independent variables in Model 4. In Model 5, the resilient modulus is the dependent

Table 24. Summary of R^2-values at the 90th percentage test value obtained for the five models

Model Number	Confining Pressure (kPa, psi)		
	13.8 (2.0)	27.6 (4.0)	41.4 (6.0)
	R^2		
1	0.91	0.87	0.92
2	0.93	0.94	0.92
3	0.94	0.94	0.96
4	0.98	0.98	0.98
5	0.90	0.90	0.94

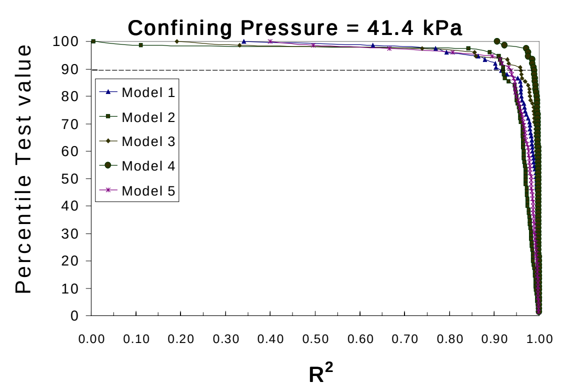

Figure 100. Percentile test value as a function of R^2 obtained from models 1 through 5.

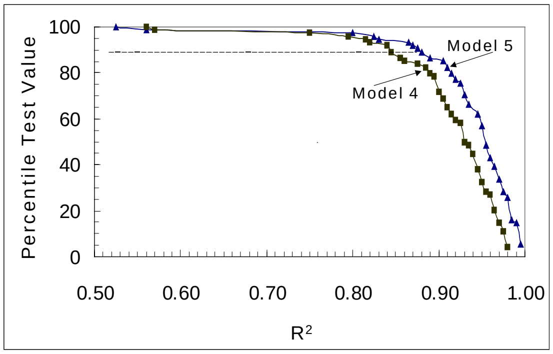

Figure 101. Comparison of R^2 results of Models 4 and 5.

variable while the deviator stress and confining stress are independent variables. Hence, the regression equations of both models represent a regression plane in a three-dimensional rectangular coordinate system. In the multiple correlation analysis of Models 4 and 5, all 15 data points were used collectively to obtain the coefficients k_1, k_2, and k_3. The coefficients for both models are presented in Appendix G. The 15 points correspond to confining stresses of 13.8, 27.6, and 41.4 kPa (2, 4, and 6 psi) and deviator stresses of, 13.8, 27.6, 41.4, 55.1, and 68.9 kPa (2, 4, 6, 8, and 10 psi). The coefficient of multiple correlation, R^2, was determined for each of the 89 tests and for each model. Percentile test value as a function of the coefficient of multiple correlation for each model is shown in Figure 101. At the 90th percentile test value the value of R^2 obtained from model 5 is about 0.88. For Model 4, the corresponding value is 0.85. At the 67th percentile test value, the values of R^2 are 0.94 and 0.88, respectively. Model 5 provides a slightly better "fit" of the relationship between resilient modulus and stresses than Model 4 for the domain of stresses used in the test.

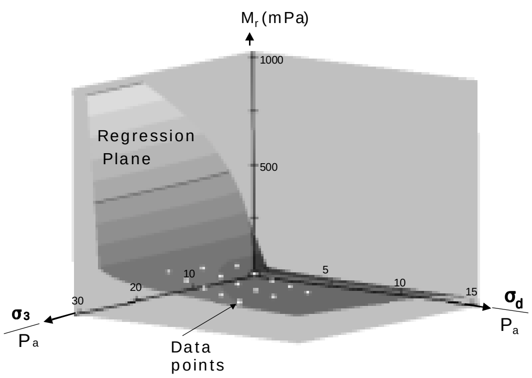

Figure 102. Least square regression plane of Model 4.

Typical views of the least square regression planes of Models 4 and 5 are shown in Figures 102 and 103, respectively. Actual data points are shown plotted on the regression planes of both models. In both cases, the points lie close to the regression planes. However, as shown in Figure 102, the regression plane, or the value of resilient modulus, of Model 4 approaches infinity as the values of stress become small, or as the values of stress approach zero. Figure 104 provides another view of this situation. However, as the stresses approach zero in Model 5, the resilient modulus does not approach infinity, as illustrated in Figure 103. The resilient modulus of the regression plane of Model 5 approaches the coefficient k_1, or the resilient modulus approaches the initial resilient modulus of the specimen as the stresses approach zero. Consequently, Model 5 appears to provide a better correlation plane than Model 4 and it does not diverge toward infinity at low stresses.

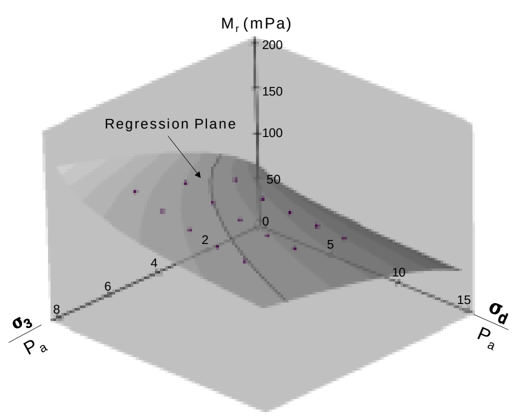

Figure 103. Least square regression plane of Model 5.

Results of Multiple Regression Analysis of Resilient Modulus Tests Performed on Untreated and Treated Subgrade Specimens

Coefficients, k_1, k_2, and k_3, obtained from multiple regression analysis using models 4 and 5

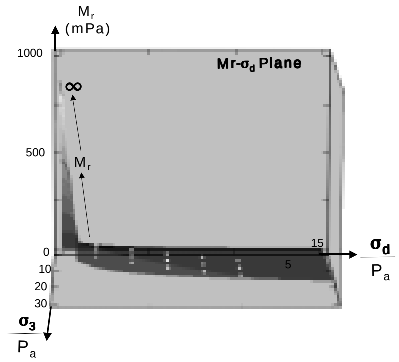

Figure 104. View of the regression plane of Model 4 in the direction of the. M_r-σ_d plane.

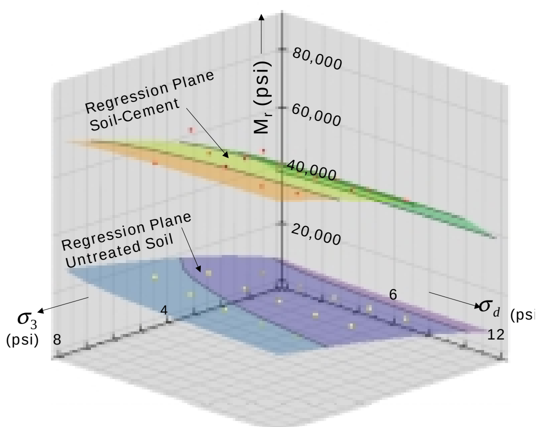

Figure 105. Examples of least square regression planes from Model 5 for soil-cement and untreated soil specimens.

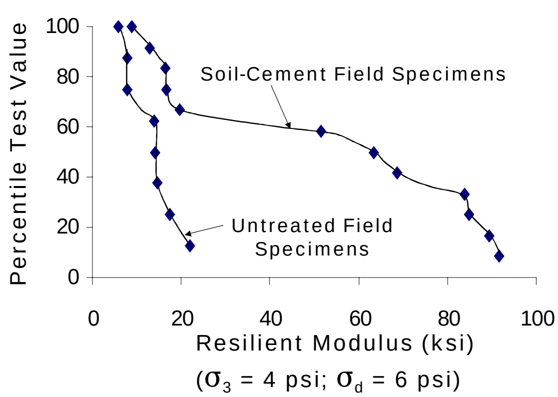

Figure 106. Percentile test value as function of resilient modulus of untreated field specimens and soil-cement field specimens.

(Equations 10 and 11, respectively) are summarized in APPENDIX G. An example of the regression planes obtained from multiple regression analyses using Model 5, Equation 11, is shown in Figure 105. In this figure, the regression planes obtained for the soil-cement subgrade specimen and the untreated subgrade specimen are compared. Both specimens were obtained at the same location. Variation of the resilient modulus with deviator stress and confining stress is illustrated in this three-dimensional graph. Actual M_r-σ_d-σ_3 data points obtained from the resilient modulus tests are compared to each regression plane predicted from the Model 5 analyses. The upper plane is the resilient modulus regression plane of a soil-cement specimen while the lower plane is the regression plane of an untreated soil specimen obtained at the same location as the soil-cement core. Values of resilient modulus of the soil-cement cores were much larger than resilient modulus values of the untreated specimens.

As one means of comparing values of resilient modulus of chemically treated and untreated specimens, resilient modulus values were calculated using the coefficients, k_1, k_2, and k_3, from Model 5, Equation 11. Deviator and confining stresses equal to 41.4 kPa (6 psi) and 27.6 kPa (4 psi), respectively, were assumed in the calculations. Those stresses are located at about the midpoint of the domain of testing stresses (and regression planes shown in Figure 105). Values of resilient modulus obtained for the untreated and soil-cement specimes are compared in Figure 106. Percentile test value is shown as a function of the resilient modulus. In all cases, the resilient modulus of the soil-cement specimens are larger than resilient modulus of the untreated specimens. Values of resilient modulus of the untreated subgrade specimens range from 6 ksi (41.36 mPa) at the 100th percentile test value to 22 ksi (151.65 mPa) at the 15th percentile test value. However, at the 100th and 15th percentile values, the resilient modulus values of the soil-cement field specimens range from about 9 to 90 ksi (62.05 to 620.46 mPa), respectivley. Values of resilient modulus of the soil-cement specimens are about 1.5 to 4.1 times larger than the resilient modulus of the unsoaked and untreated field specimens.

Values of resilient modulus of soil-hydrated

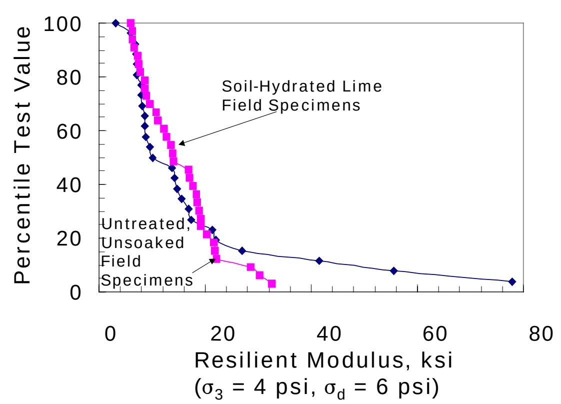

Figure 107. Percentile test value as function of resilient modulus of untreated and soil-hydrated lime field specimens.

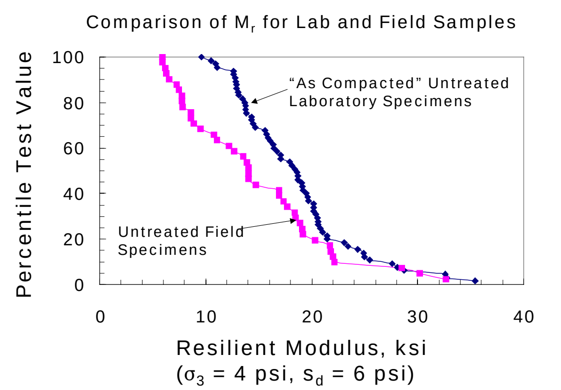

Figure 108. Percentile test value as a function of resilient modulus of untreated and soil-cement field specimens

specimens and untreated, unsoaked field specimens are compared in Figure 107. In both series of specimens, the values of resilient modulus are fairly large. Basically, values of resilient modulus of the two different series of specimens are nearly equal from about the 95th to 20th percentile test value and range from about 6 ksi to 22 ksi (41.36 to 151.65 mPa). Values of resilient modulus of the soil-hydrated lime specimens ranged from 22 to 60 ksi (151.65 to 413.58 mPa) between the 20th and 5$^{th.}$ percentile test values. Past testing (Hopkins et al, 1985) has shown that clayey soils, when first compacted and not subjected to soaking, have CBR values that range from about 10 to 45. However, when the same clayey soils are soaked, the CBR values generally range from about 1 to 6. Accordingly, it could be expected that values of unsoaked specimens would be larger than values of resilient modulus of soaked specimens.

The untreated field specimens were obtained below the " soft zone" of untreated soil. These specimens were unsaturated (or unsoaked) and their resilient modulus behavior is similar to the resilient modulus behavior of "as compacted" (unsaturated) specimens. To illustrate, the resilient modulus of field specimens are compared in Figure 108 to resilient modulus of recompacted (Kentucky) clayey soils of all types (Hopkins et al, 2002). Assuming deviator and confining stresses equal to 6 psi and 4 psi (41.4 to 27.5 kPa), respectivley, values of resilient modulus were computed using the regression coefficients of model 5 (Equation 11). The laboratory data in this figure represent the results of about 72 resilient modulus tests that were performed on unsoaked, or "as compacted," and untreated specimens (Hopkins et al 2002). Values of resilient modulus of the laboratory specimens ranged from about 9.4 to 26 ksi (64.79 to 179.22 mPa) at the 100th and 10th percentile test values, respectively. Values of resilient modulus of the field specimens were only slightly lower than the resilient modulus values of the laboratory (unsoaked) compacted specimens, as illustrated in Figure 108. Values of resilient modulus of the field specimens ranged from about 6 ksi to 26 ksi (41.35 to 179.22 mPa) at the 100th and 10th percentile test values, respectively.

In the same study (Hopkins et al 2002), the same clayey soils as those used to form "as compacted" laboratory specimens were remolded to identical dry densities and moisture contents. In that series, (60) specimens were allowed to soak for 2 to 4 weeks. After swelling had completely ceased, resilient modulus tests were attempted. Unfortunately, resilient moulus tests generally could not be performed following the AASHTO T-294[4] standard because of large deformations of the

[4] Research is in progress to develop a resilient moulus testing procedure for soaked specimens.

saturated specimens. The saturated specimens usually deformed to such a degree that testing had to suspended because the strains were outside the range of the LVDTs. When values could be obtained, the resilient modulus values were generally much less than 6 ksi (41.36 mPa). Typically, the values ranged from 1.8 to 3.2 ksi (12.41 to 22.06 mPa). However, these values were probably lower than the actual values because the initial height of the specimen changed so much during the test that the caluculations of resilient modulus were affected. The "permanent set' after each testing sequence could not be monitored. As shown in Figure 85, the moisture contents of these specimens were much smaller than moisture contents of the soil at the top ("soft zone") of the untreated subgrade. Hence, the resilient modulus of the unsoaked specimens would be much higher than the soils at the top of the subgrade in the soft zone of the subgrade. The fact that no failures occurred in the resilient modulus testing of untreated field specimens was another strong indication that the field specimens were unsoaked and unsaturated.

Falling Weight Deflectometer (FWD) Measurements and Back Calculations

Falling weight deflectometer (FWD) tests were performed on all pavement sections selected for testing. Personnel of the Kentucky Transportation Center's Pavement Section performed the tests. Test data were reduced using MODULUS 5.0 developed by the Texas Transportation Institute, (Michalak and Scullion, 1995). Values of modulus were calculated for all pavement components (asphalt, stone base, stabilized subgrade, and the subgrade below the stabilized layer). Average modulus values for each section and each pavement component are shown in Table 25. Values of thickness used in calculating the modulus values of the asphalt were determined by directly measuring asphalt concrete cores. The stone base thickness was measured in boreholes advanced through the asphalt concrete and stone base to the subgrade. Stabilized subgrade thickness was measured from standard penetration test samples.

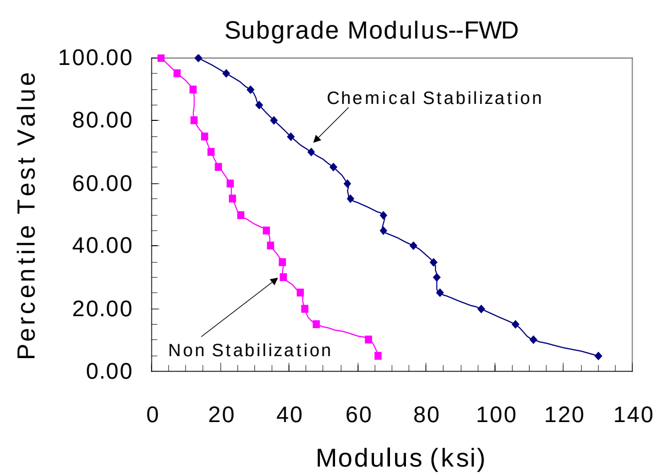

Figure 109. Percentile test value for stabilized and non-stabilized subgrades as a function of modulus obtained from FWD measurements and calculations.

As shown in Table 25, values of modulus of the chemically stabilized subgrades are much higher than the non-stabilized subgrades situated below the stabilized layers. This situation is also illustrated in Figures 109 and 110.

Chemical admixtures used in the stabilized subgrade included Portland cement, hydrated lime, lime kiln dust, and a hydrated lime-cement combination. Values of modulus, obtained from FWD measurements and back-calculations, of the stabilized layers range from about 21.6 to 130 ksi (148.89 to 896.09 mPa). Values of modulus of the non-stabilized subgrades ranged from about 2.7 to 66.1 ksi (18.61 to 455.63 mPa). Average modulus values for crushed stone base aggregates (limestone) ranged from 29 to 231.7 ksi (199.90 to 218.51 mPa). The values of modulus of the granular bases resting on stiff layers of stabilized subgrades are generally much higher than values of modulus of granular bases resting on soft, soil subgrade layers (Southgate et al 1981).

Table 25. Average modulus values of study sections and pavement components obtained from FWD measurements and backcalculations.

County/Route	**Layer Thickness/Type of Layer/Modulus (psi)**			
				Untreated
Anderson -US 127	13" AC- 1,061,000	4" Drainage Blanket--------- 92,300	12" Lime Stabilized Subgrade-- 52,900	Subgrade--12,000
Boyle-- US 127 By Pass	14" AC- - 1,118,000	4" Drainage Blanket——------ 110,000	12" Lime Stabilized Subgrade-- 57,100	Subgrade--7,500
Fayette--US 127	12" AC—1,721,000	8" Dense Graded Aggregate— 29,000	10" Lime Stabilized Subgrade-- 28,700	Subgrade—12,000
Hardin--US 127	11" AC--1,380,400	4" Crushed Stone Base--84,000	8" Lime Stabilized Subgrade- 67,600	Subgrade-38,400
Shelby--KY 55	11" AC--2,023,000	None	8" Lime Stabilized Subgrade- 21,600	Subgrade--2,700
Owen--US 127	10" AC—1,062,000	4" Crushed Stone Aggregate— 40,200	8" Lime Stabilized Subgrade-- 46,400	Subgrade—12,300
Trigg--US 68	10" AC—1,414,000	6" Crushed Stone Aggregate- 189,000	12" Lime Stabilized Subgrade— 130,100	Subgrade—15,500
Boone--KY 842	9" AC—890,000	5" Dense Graded Aggregate— 80,000	12" Lime- Cement Stabilized Subgrade 67,400	Subgrade—19,400
Breckinridge--US 60	10" AC—1,299,000	5" Dense Graded Aggregate— 38,300	16" Cement Stabilized Subgrade— 31,400	Subgrade-38,000
Daviess--KY 331	8" AC—726,000	None	22" Cement Stabilized Subgrade— 58,000	Subgrade—17,200
McCracken--US 62	8" AC—838,500	4" DGA/12" Bank Gravel - 107,100	12" Kiln Dust Stabilized Subgrade— 105,800	Subgrade—22,800
Hickman--US 51	9" AC—856,500	6" Bank Gravel—122,900	15" Kiln Dust Stabilized Subgrade— 111,000	Subgrade—23,400
McCreary--US 27	8" AC—1,049,000	4" Crushed Stone Base—130,900	12" Cement Stabilized Subgrade— 82,000	Subgrade—26,000
Lee--KY 11	9" AC—2,474,000	5" Crushed Stone Base--64,400	12" AFBC Stabilized Subgrade— 35,700	Subgrade—43,300
Lee--KY 11	7" AC—2,500,000	5" Crushed Stone Base—231,700	12" Cement (10%) Stabilized Subgrade—95,900	Subgrade—66,100
Lee--KY 11	9" AC --2,500,000	5" Crushed Stone Base—108,800	12" Lime Stabilized Subgrade— 76,100	Subgrade—63,100
Lee--KY 11	9" AC—1,946,000	4" Crushed Stone Base—184,600	12" Kiln Dust Stabilized Subgrade— 83,100	Subgrade—44,000
Lee--KY 11	7" AC—2,126,000	5" Crushed Stone Base--186.900	12" Cement (7%) Stabilized Subgrade— 84,000	Subgrade—48,100
Lee--KY 11	10" AC—2,244,000	6" Crushed Stone Base—62,700	None ---	Subgrade—33,500
Lee--KY 11	9" AC—2,226,000	6" Crushed Stone Base—58,100	12" AFBC Stabilized Subgrade— 40,600	Subgrade-34,800

That condition is clearly illustrated in Figure 111. The average FWD modulus (of each section) of DGA or crushed stone base is shown as a function of the average FWD modulus of soil-cement and soil-hydrated lime subgrades. As the FWD modulus of the soil-cement and soil-hydrated lime subgrades increase, the FWD modulus of the DGA, or crushed stone, base increases. At a value of 27 ksi (186.11 mPa) of the treated subgrade, the two curves converge at a granular base modulus of 19.6 ksi (135.10 mPa). This point of intersection may represent a "threshold value of modulus" of the granular base. If the modulus of the material supporting the

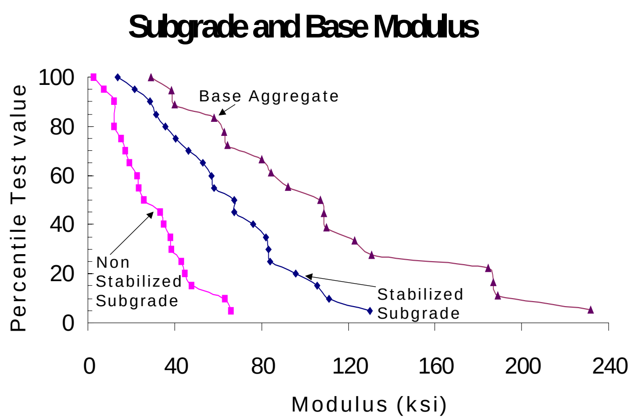

Figure 110. Comparison of average values of FWD modulus of base aggregate, chemically stabilized subgrades, and non-stabilized subgrades.

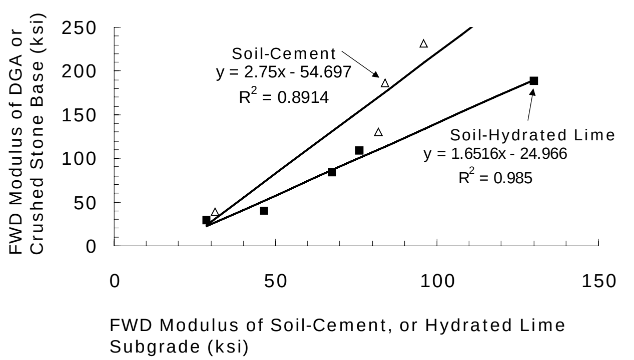

Figure 111. Average FWD modulus of DGA or crushed stone base as a function of the average FWD modulus of soil-cement and soil-hydrated lime subgrades.

granular base decreases to a value lower than about 27 ksi (186.11 mPa), then the value of modulus of the granular base may not decrease below the value of about 19.6 ksi.

Based on a correlation published by AASHTO (1993)[5], the structural layer coefficient, a_2, of a granular base is estimated to be 0.14 at a (resilient) modulus value of 30 ksi (206.79 mPa). Based on the AASHTO correlation (which has been recast in the form shown in Figure 112), the a_2-structural layer coefficient increases as the modulus of the granular base increases. When the FWD modulus of either the soil-cement, or soil-hydrated lime subgrade is equal to 27 ksi, the FWD modulus of the aggregate base is the same regardless of the type of stabilized subgrade. However, when the FWD modulus of the stabilized subgrade is greater than 27 ksi, as shown in Figure 110, the modulus of the granular base resting on the soil-cement subgrade is greater than the modulus of the granular base resting on the soil-hydrated lime subgrade. For example, when the FWD modulus of the treated subgrade is 100 ksi (689.30 mPa), then the modulus of the aggregate base resting on the soil-cement is 220 ksi (1,516.46 mPa) while the modulus of the aggregate base resting on the soil-hydrated lime subgrade is 140 ksi (965.02 mPa). Basically, as the stiffness of the stabilized subgrade increases, the modulus of the aggregate base increases. Conversely, as the modulus, or stiffness, of the subgrade decreases, the modulus of the aggregate base decreases. Typically, CBR-values of clayey subgrades in Kentucky range from 1 to 6. Estimated values of modulus of those subgrades are 1 to 9 ksi (6.89 to 62.03 mPa)(Hopkins et al 2002)—values that are much lower than values of modulus obtained for soil-cement and soil-hydrated subgrades. Hence, it could be expected that the modulus of aggregate bases resting on the very soft clayey soil subgrades in Kentucky would be very low and much less than values of modulus of aggregate bases resting on soil-cement and soil-hydrated lime subgrades.

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[5] "Figure 2.6. Variation in Granular Base Layer coefficient (a_2) with Various Base Strength Parameters....." (AASHTO 1993).

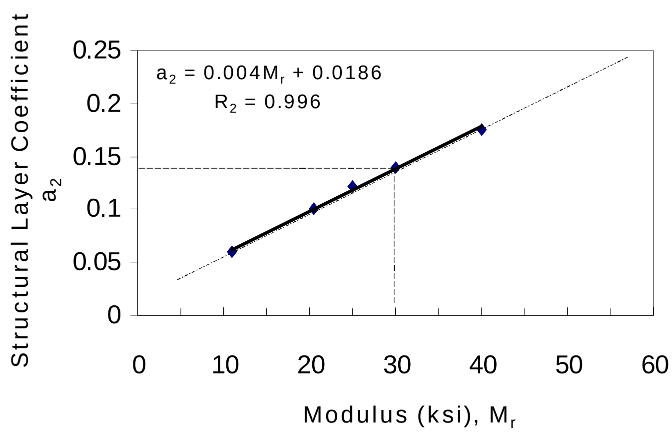


Figure 112. Relationship between the structural layer coefficient, a_2, and modulus (after AASHTO 1993).

As the modulus of the either the soil-cement subgrade or soil-hydrated lime subgrade increase above the threshold value of 27 ksi (186.11 mPa), the modulus of the granular base increases. Consequently, as illustrated in Figure 112, when the modulus of the granular base increases, the structural layer coefficient of the granular base increases. According to Figure 112, for instance, as the modulus of the granular base increases from 19.6 ksi (130.97 mPa) to a value of 50 ksi (344.65 mPa), the a_2-structural layer coefficient increases from 0.10 to about 0.22, respectively. Hence, by using chemically stabilized subgrades, which have much higher values of modulus and stiffness than untreated subgrades, the overall structural integrity of the pavement is improved.

More recently (AASHTO 2002, NHCRP Project 1-37A) presents a relationship, Figure 113, between resilient modulus and the layer coefficient, a_i, as

$$M_r = 30{,}000\left(\frac{a_i}{0.14}\right)^3 \quad \text{(psi)}, \tag{16}$$

where a_i is the experienced-based layer coefficient of a given agency for base and subbase layers. If the modulus of the treated soil-cement layer increases from 27 to 100 ksi (186.11 to 689.30 mPa), the modulus of the granular base increases from 19.6 to 220 ksi (135.10 to 1,516.46 mPa). Based on equation 16, the layer coefficient increases from 0.122 to about 0.272. When the modulus of the soil-hydrated lime layer increases from 27 to 100 ksi (186.11 to 689.30 mPa), then the modulus of the granular base increases from 19.6 to 140 ksi (135.10 to 965.02 mPa) and the layer coefficient increases from 0.122 to about 0.235. Regardless of which curve is used, Figure 112 or Equation 16 (Figure 113), the modulus of the granular base increases as the modulus of the chemically stabilized layer increases.

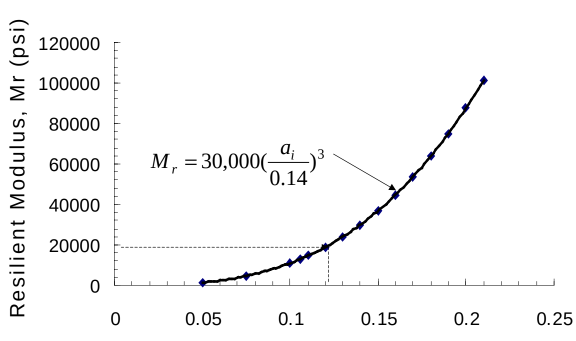


Figure 113. Relationship between resilient modulus and layer coefficient, a_i (after NCHRP Project 1-37A, 2002 Milestones, Fall 2001).

Rutting Measurements of Pavement Sections

Rutting measurements were made every 500 feet in each section. As shown in Table 26, average rutting depths of the sections at the 50^{th} and 20^{th} percentile test values ranged from 0.11 to 0.29 inches (0.28 to 0.74 cm) and 0.16 to 0.37 (0.41 to 0.94 cm), respectively. The average values of rutting depths at all sites were 0.20 and 0.27 inches

Table 26. Average rutting measurements.

Roadway Site	Average Rutting Depth at the 50th percentile test value (inches)	Average Rutting Depth at the 20th percentile test value (inches)
Anderson	0.21	0.27
Boyle	0.16	0.23
Fayette	0.25	0.30
Lee	0.12	0.20
McCreary	0.12	0.21
Shelby	0.28	0.37
Hardin	0.22	0.28
Owen	0.23	0.28
Trigg	0.11	0.16
Boone	*	*
McCracken	0.26	0.31
Hickman	0.20	0.30
Breckenridge	0.29	0.31
Daviess	**	**

** No measurements--asphalt overlay constructed after about 15 years.*
*** No measurements--asphalt overlay constructed after about 15 years.*

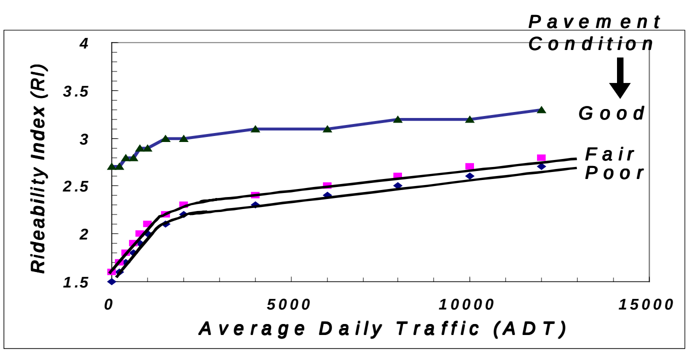


Figure 114. Rideability index as a function of the average annual daily traffic and condition of pavement.

(0.51 and 0.69 cm), respectively. Considering that the ages of the sites ranged from about 7 to 15 years, the rutting depths were generally considered to be small.

Evaluation of Pavement Conditions of Study Sections

Although detailed pavement condition assessments are not performed on all pavements in Kentucky, rideability indices are obtained for all state-maintained pavements (Burchett[6] 2001). According to Burchett, past experience and analyses of rideability indices, AADT, and subjective assessments of surfacing conditions have indicated that the need for resurfacing are closely associated with some critical rideability index (RI). When the RI-value of a pavement is below the critical RI-value, which is based on traffic volumes, the pavement is considered in poor condition and may require rehabilitation, or at the minimum, a closer inspection to determine the condition of the pavement. Critical relationships between critical RI-values and traffic volumes are defined in Figure 114[6].

As one means of estimating the conditions of the study sections, RI–values were obtained for the highway sections from published records of KYTC. Trend relationships of RI-values as a function of time were developed for each study section when data was available. A typical relationship of

[6] Table 3 in a draft report entitled “Pavement Management in Kentucky: An Overview in Year 2001,” Pavement Management Branch of the Kentucky Transportation Cabinet, February 2001—Private communication with Jim Burchett, former Branch Manager.

RI as a function of time is illustrated in Figure 33. Based on the trend relationships, values of RI occurring at the time of the study were computed. Also, predicted values of RI at times of 15 and 20 years after construction were estimated from the trend relationships. Assessments of the conditions of the study pavement sections were made using the assessment curves in Figure 114. Values of RI and AADT for the pavement lives at the time of the study, 15 years, and 20 years and corresponding predicted AADT values are summarized in Table 27. Using the RI-values and the average annual daily traffic (AADT), the condition of each pavement section was estimated from the curves in Figure 114. The condition assessments are summarized in Table 28. Conditions of all pavement sections at the time of the study and projected times of 15 and 20 years were rated as "good" except for sections identified as US 127 (Owen County), US 25-42 connector (Boone County), and KY 331 (the River Port Access Road). Based on initial RI-measurements of the US 127 roadway pavement shortly after construction, the pavement was rated as "good". At the time of the study, this rating had

Table 27. Summary of Rideability Indices and values of AADT of the study sections.

County	Route Number	Age	$RI_{in.}$	RI_{age}	RI_{15}	RI_{20}	$AADT^{2}_{age}$	$AADT_{15}$	$AADT_{20}$
Anderson	US 127	7	4.00	3.63	3.37	3.20	6,510	7,242	7,833
Boyle	US 127 Bypass	9	4.00	4.00	4.00	4.00			
Fayette 16.970-17.000 16.374-16.970	US 25	6	*	3.60	RI[1]	RI[1]	15,800-17,600	20,300-22,086	21,800-25,478
Lee: KY 11	Sections 1 through 7	12	3.65 to 3.56	3.51 to 3.31	3.49 to 3.28	3.48 to 3.25	2,550	2,717	2,934
McCreary	US 27	10	3.77	3.60	3.52	3.43	6,400	6,833	7,373
Shelby	KY 55	8	3.40	3.34	3.29	3.25	15,200	25,286	32,470
Hardin	US 62	10	3.8	.67-3.61	3.64 - 3.57	3.62-3.55	6,360-9,640	7,578-13,129	8,501-16,012
Owen	US 127	8	3.72	2.73	1.87	1.67	2,330	2,591	2,834
Trigg	US 68	3.90	4.00	3.90	3.90	3.90	9,390	11,156	12,137
Boone	KY 842 US 25-42 Connector	11	3.57	3.27[3]			6,850	11,642	13,095
McCracken	US 62	10	3.77	3.63	3.61	3.59	8,910	11,647	13,095
Hickman	US 51	8	3.40	3.50	3.50	3.50	2,440	2,096	3,571
Breckinridge	US 60	13	3.80	3.70	9	4.0	3,290	3,527	4,113
Daviess	KY 331 (River Port Access Road)	13	Na[4]				6,620	6,818	7,384

* *RI data obtained from KYTC, Division of Operations, Pavement Management Branch shows a construction date of 1980. However, published data for 1994 shows a construction date of 1994 and value of RI of 3.6.*
1. Insufficient RI data to establish a trend line. 2. Average Annual Daily Traffic at the time of the study, or age of the section. 3. A thin overlay was constructed 10 years after construction—the RI value of the section was 3.0 before the construction of the overlay. *4. No RI-values published for this roadway section.*

decreased to "fair." At projected times of 15 and 20 years, the pavement would be rated as poor. However, the section was designed for only 600,000 ESALs and more than 70 percent of the design life had been used at the time of the study. At a projected year of 2002 (about 11 years after construction), the design life of this pavement section will have been used. In the case of the US 25-42 connector in Boone County, about 79 to 100 percent of the design life of this pavement has been used. Hence, the rated condition of "fair" at the time of this study, and a predicted rating of "poor" at

Table 28. Summary of back-calculated values of the layer coefficient, a_3, of treated subgrades.

		Pavement Condition			
County	**Route Number**	**$AADT_{initial}$**	**$AADT_{age}$**	**$AADT^{1}_{15}$**	**$AADT^{1}_{20}$**
Anderson	US 127	good	good	good	good
Boyle	US 127 Bypass	good	good	good	good
Fayette	US 25	good[2]	good	good	good
Lee	KY 11	good	good	good	good
McCreary	US 27	good	good	good	good
Shelby	KY 55	good	good	good	good
Hardin	US 62	good	good	good	good
Owen	US 127	good	fair	poor	poor
Trigg	US 68	good	good	good	good
Boone	KY 842 (US 25-42 Connector)	good	Fair- good[3]	--	--
McCracken	US 62	good	good	good	good
Hickman	US 51	good	good	good	good
Breckinridge	US 60	good	good	good	good
Daviess	KY 331 (River Port Access Road)	***	***	***	***

1. Projected values of AADT from trend relationship of AADT and time.
2. RI-values obtained from the Kentucky Transportation Cabinet, Division of Operations, Pavement Management Branch shows a construction date of 1980. However, published data for 1994 shows a construction date of 1994 and a value of RI of 3.6.
3. A thin overlay was constructed about 15 years after construction—the RI-value was 3.0 before the overlay and, based on this RI-value, the pavement would have been assessed as " fair to good."
**** No RI data available for this access road. However, an overlay was consrtucted near the end of this study at an age of about 15 years.*

projected times of 15 and 20 years, would be expected. After 15 years of service, a thin overlay was constructed at the Boone County site. The RI-value before placement of the overlay was reportedly 3.0, and based on this value, the pavement condition would have been rated as "fair to good."

As another means of assessing the ride quality of the test sections, evaluations of the sections were made using the Kentucky Transportation Cabinet's "ride quality adjustment schedule" that is used to adjust the pay to contractors for new pavements. The pay value is adjusted upward or downward according to the rideability index of the newly constructed payment. The data appearing elsewhere (see last footnote) is shown in the form of graphs in Figure 115. The rideability of the new pavement is shown as a function of the pay adjustment value (plus or minus percentage). As shown in this figure, if the RI-value of the new pavement is below a value of 3.45, then the new pavement must be corrected or redone. When the RI-value of the new pavement ranges from 3.45 to 3.60 the pavement does not have to be corrected, but there is a 15 percent reduction in the contractor's payment. If the RI-value exceeds 3.60, then the payment is increased, as shown in Figure 115.

Based on the initial values of RI, the Shelby County site would have been marginal since the initial RI value was 3.4 or slightly below the acceptable value of 3.45. The RI values of all other sections were greater than 3.45, except the Daivess County site. Since RI data was not available for the Daviess County site, no evaluation could be performed.

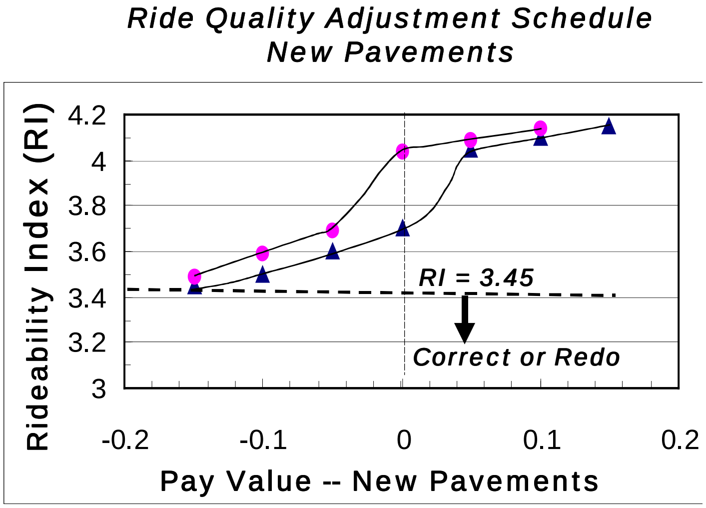
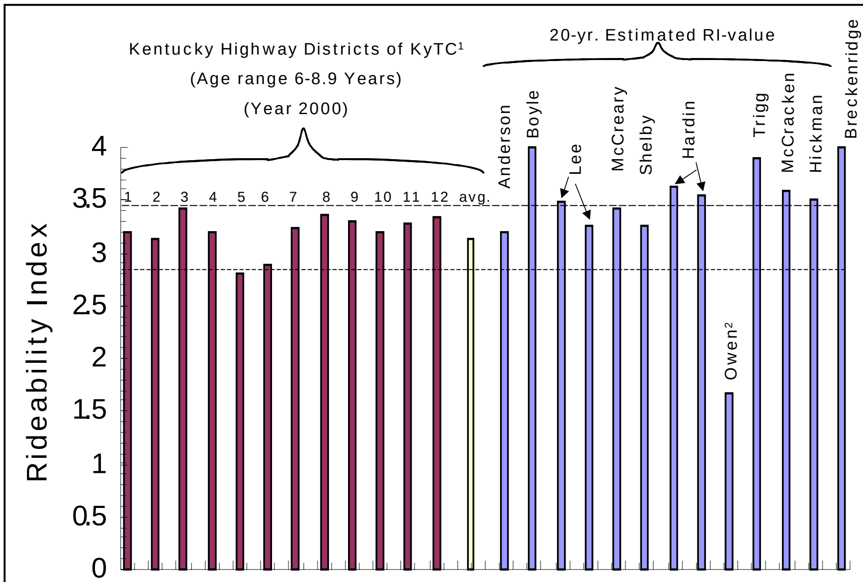


Figure 115. Rideability index as a function pay value used to reward pavement contractors for constructing pavements with good riding qualities.

In the left-hand portion of Figure 116, average values of RI for pavements in each highway district of the Kentucky Transportation Cabinet are shown for the year 2000 (Burchett, 2001). Ages of the pavements ranged from 6.0 to 8.9 years at the time of the RI measurements. The RI values ranged from 2.80 to 3.42. RI-values, which are based on a projected 20-year trend and appear in the right-hand portion of Figure 116, are compared to the average RI-values (left-hand portion of the figure) of the district pavements. The comparison shows that, generally, the 20-year projected RI-values of most of the chemically-treated subgrade sections (15 of 20 sections) were much higher than the average district RI-measurements. In the case of the Daviess County, no RI data had been published and no analysis could be performed—an AC overlay was constructed after about 15 years. At the Fayette County site, insufficient data was available to develop a trend relationship of RI and time. At the time of the study, the value of RI was 3.6 at that site. At the Boone County site, the ESAL life of the pavement had been used and a thin overlay had been constructed after about 15 years.



*NOTES: 1. Ages of pavements in districts 1,2,3,4,5,6,7,8,9,10, 11, and 12 at the time of RI measurements were 8.5, 8.9, 8.6, 6.1, 6.0, 7.2, 8.9, 7.3,7.1, 7.1, and 6.6 years, respectively. 2. Seventy % of ESAL life expired at time of study (thin pavement. *Not shown: Fayette Rt. US 25 -- Insufficient data to project RI-time trend -at time of study, RI = 3.60 ; Boone (RT. Ky 842) RI data not shown---thin overlay after 15 years--ESAL Life used; Daviess-- RI data not published.*

Figure 116. Average rideability index and age for the twelve Districts Offices of the Kentucky Transportation Cabinet and the year 2000.

Economical Analysis

Most of the pavement sections selected for this study were originally designed to be located on non-stabilized subgrades. In those designs, the assumption was made that the pavements were to rest on the native, compacted soil subgrades. However, the subgrades were stabilized and actually contribute to the structural integrity of the pavement. In the early development of the stabilization program, the chemically stabilized subgrade was frequently treated as a "working platform" so that the pavement could be constructed without problems. However, at a few of the roadway sections, thicknesses of the pavements were slightly thinner than thicknesses obtained from the 1981 Kentucky design method.

As shown previously in Table 19, the structural layer coefficient, a_3, of the subgrades stabilized with cement, hydrated lime, or combinations of hydrated lime and cement, and lime kiln dust, AFBC ranges from 0.10 to 0.127. If all three pavement components--AC, base, and stabilized subgrade—and the stabilized coefficients in Table 19 are used, as well as the actual thicknesses of the pavement components, to determine the in place structural number, $SN_{inplace}$, then the in-place structural number is greater than (or equal to) the structure number, $SN_{required}$, required, by the 1981 Kentucky design curves when the design values of ESAL and subgrade CBR are used. If no structural credit was given to the stabilized subgrade, then the values of $SN_{nonstab}$ at eight sections would have been less than the structural number, $SN_{required}$, required by the 1981 Kentucky design curves.

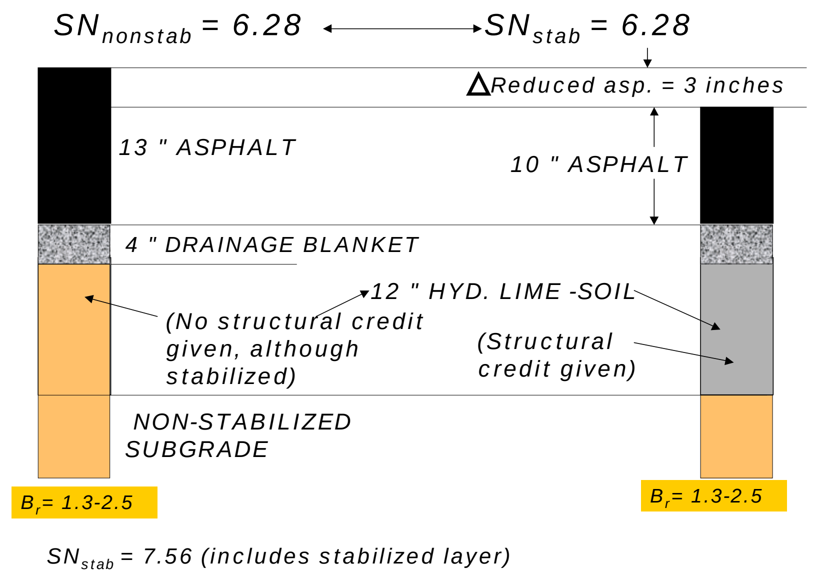


Figure 117. Pavement scenarios used in the economical analysis.

Since the design situations varied at different sections, the economical analysis of chemically stabilized subgrades was based on the structural number, $SN_{required}$, required to satisfy the Kentucky design curves. Two different scenarios were analyzed, as depicted in Figure 117. Using the design values of ESAL and the non stabilized, subgrade CBR for each study section, the structural layer number, $SN_{required}$, at each site was determined from the 1981 Kentucky design curves. In the first case, $SN_{required}$ may be expressed as a function of the structural layer coefficients, a_1 and a_2, and the required design thicknesses, $d_{1nonstab}$ and d_2, of the asphalt concrete (AC), and the granular base, respectively, or

$$SN_{required} = a_1 d_{1nonstab} + a_2 d_2 \, . \tag{17}$$

In using the 1981 Kentucky design curves, the percentage of AC must be assumed to determine the value of $SN_{required}$. To maintain compatability in the approach used in the analyses, the ratio of the thickness of the AC layer to the total design thickness, expressed as a percentage, was obtained by using the actual measured in place thickness of the AC and base, or

$$AC_{percentage} = \frac{AC_{thickness}(100)}{AC_{thickness} + base_{thickness}} \quad . \tag{18}$$

After determining $SN_{required}$, the design thickness of AC, $d_{1nonstab}$, was determined by solving Equation 17, or

$$d_{1nonstab} = \frac{SN_{required} - a_2 d_2}{a_1} \; . \tag{19}$$

Assuming the structural layer coefficients of AC and granular base are equal to 0.44 and 0.14, respectively, and using the measured thickness of granular base, Equation 19 becomes

$$d_{1nonstab} = \frac{SN_{required} - (0.14)(base_{thickness})}{(0.44)} \tag{20}$$

In the second scenario, shown in the right-hand portion of Figure 117, calculations were performed to determine the thickness, d_{1stab}, of AC that is needed when structural credit is given to the stabilized subgrade. The same structural layer number, $SN_{required}$, of each section obtained in the first scenario may be expressed, as follows:

$$SN_{required} = a_1 d_{1stab} + a_2 d_2 + a_3 d_{stablayer} \; . \tag{21}$$

Where

- a_1, a_2 = structural layer coefficients of AC and base = 0.44 and 0.14, respectively,
- d_{1stab} = thickness of AC when structural credit is given to the stabilized subgrade,
- d_2 = thickness of base measured during field operations,
- a_3 = structural layer coefficient of stabilized subgrade at the 85th percentile test value (see Table 19), and
- $d_{stablayer}$ = thickness of stabilized layer measured during field operations.

The thickness of AC required when stabilization is used may be obtained by solving Equation 21, or

$$d_{1stab} = \frac{SN_{required} - a_2(d_2) - a_3(d_{stablayer})}{a_1} = \frac{SN_{required} - (0.14)(base_{thickness}) - a_3(d_{stablayer})}{0.44} . \tag{22}$$

When structural credit is given to the subgrade, the required thickness of AC, or granular base, may be reduced. Thicknesses of the bases used in the study sections were very thin and the minimum values of thickness were generally used. Consequently, the assumption was made that, if one of the pavement components was reduced in thickness, than it would be the AC layer, or

$$\Delta d_{1AC} = d_{1nonstab} - d_{1stab} \; . \tag{23}$$

Hence, the cost of the reduced thickness, Δ_{1AC}, of the AC layer could be compared to the cost of building the chemically stabilized layer. Unit costs assumed in making the economical analysis are summarized in Table 29. The costs are 2001 average values from records of the Kentucky Transportation Cabinet. It was assumed that if the AC layer was reduced, then only the AC base layer would be reduced. Hence, unit cost of the AC base was used in the analyses. Cost data for Lime Kiln Dust and AFBC chemical stabilization were not available. The assumption was made that the unit cost of these stabilization methods would be about 85 % of the unit cost of hydrated lime stabilization since those materials are byproducts and were either purchased at a cheaper price than hydrated lime or donated, as it was in the case of the AFBC ash.

Table 29. Unit costs used in the economical analysis of chemical stabilization*.

Pavement Layer	Unit Costs (dollars)	Unit Cost Based on:
Asphalt Base	1.87 yd^2/in.depth	34.18 ton: Asphalt Base
Hydrated Lime-Soil	0.3525 yd^2/in.depth	96.13 ton: Hydrated Lime $ 1.71 yd^2 : Hydrated Lime stabilized roadway 299.38 ton: Curing Seal 22.96 ton: Blotter Sand
Cement-Soil	0.49 yd^2/in. depth	89.94 ton: Portland Cement $ 1.47 yd^2 : Cement stabilized roadway 22.96: Blotter Sand
Lime Kiln Dust-Soil	0.30 yd^2/in. depth	Assumption: Unit cost = 85 % of unit cost of hydrated lime stabilization
AFBC-Soil	0.30 yd^2/in. depth	Assumption: Unit cost = 85 % of unit cost of hydrated lime stabilization

**Unit Costs are average values for the year 2000 obtained from the Kentucky Transportation Cabinet.*

A summary of the economical analysis of using chemical stabilization for each study section is summarized in Table 30. Design values of ESAL and subgrade, as well as the percentage of asphalt concrete—based on field measurements—to the total pavement thickness are shown for each section. Measured thicknesses of the stabilized subgrades and the backcalculated coefficients, a_3, are shown. The actual structural number—excluding the structural credit of the stabilized layer --SN_{actual} , based on measured thicknesses of AC and granular base, are compared to the required design structural layer number, $SN_{required}$. In some cases, SN_{actual} is less than $SN_{required}$. In those cases, the designer may have given structural credit to the stabilized layer. Thickness of AC, $d_{1nonstab}$, obtained when no stabilization was used is compared to the AC thickness, d_{1stab}, obtained when stabilization is used. The cost of the difference, Δd_{1AC}, expressed by Equation 21, may be expressed as

$$Cost \quad of \quad \Delta d_{1AC} = (unit \quad \cos t)(\Delta d_{1AC}) \, ,$$

or, in terms of the cost per yd^2

$$Cost \quad of \quad \Delta d_{1AC} = (unit \quad \cos t)(\Delta d_{1AC} \ \ in.) = (\frac{dollars}{yd^2 \ in.})(\Delta d_{1AC} \ \ in.) = (\frac{dollars}{yd^2}) \ .$$

Similarly, the cost of stabilization of each section per yd^2 may be determined from the following relationship

$$Cost \quad of \quad Stabilization = (unit \quad \cos t)(d_3 \ \ in.) = (\frac{dollars}{yd^2 \ in.})(d_3 \quad in.) = (\frac{dollars}{yd^2}) \ .$$

The term, d_3, is the depth of the stabilized layer, which was measured during the field operations. The unit costs of AC and soil-hydrated lime, soil-cement, soil-kiln dust, and soil-AFBC stabilization are shown in Table 29.

The costs of the AC reduction and subgrade stabilization in dollars per yd^2 for each section are compared in Table 30. Costs of the AC reduced thickness range from about 3.38 to 11.87 dollars per yd^2. Costs of the subgrade stabilization range from 2.64 to 10.78 dollars per yd^2. Based on the SN value required by the 1981 Kentucky design curves, the costs of pavement sections constructed on stabilized soil subgrades are less than equivalent pavement sections constructed on non-stabilized soil subgrades, as shown in Table 30. The savings per yd^2 at a selected site is the difference in the cost per yd^2 of the pavement section with reduced thickness of AC and the cost of stabilization at a selected site, or

$$Savings(\frac{dollars}{yd^2}) = \cos t \quad \Delta d_{1AC} (\frac{dollars}{yd^2}) - \cos t \quad of \ \ stabilization(\frac{dollars}{yd^2}) \ .$$

The savings in costs per yd^2 are summarized in the right-hand portion of Table 30. Savings in unit cost range from \$ 0.48 to \$ 1.68 per yd^2 at all of the sections where subgrade stabilization had been used. The average value for all sections was \$0.96 per yd^2. By reducing the AC thickness at a selected section, the average costs in pavement savings of subgrade sections stabilized with hydrated lime and Portland cement were \$1.06 and \$0.71 per yd,^{2} respectively, of pavement surface. The average costs in pavement savings of the sections where lime kiln dust and AFBC were estimated to be \$1.23 and \$0.83 per yd^2 , respectively.

In terms of the savings per mile of roadway, and assuming the flexible pavement is 36 feet in width, the average cost is

$$Cost \quad of \quad Stabilization = (unit \cos t)(5{,}280 \ ft.)(36 \ ft)(\frac{yd^2}{9 \ ft^2}) = (unit \cos t \frac{dollars}{yd^2})(21{,}120 \, yd^2) \ .$$

The cost savings for the roadway sections are summarized in the right-hand portion of Table 30. Values range from \$10,233 to \$35,455 for all sections. Roadway savings of the pavement sections containing hydrated lime, Portland cement, lime kiln dust, or AFBC stabilized subgrades, where the AC thicknesses are reduced, are estimated to be \$22,414, \$15,080, \$25,872, and \$17, 530 per mile of roadway per 36 feet in width.

Table 30. Summary of economical cost analysis.

			Measured Thickness					Structural Layer Number								
Site	Design ESALs (Mil.)	Design CBR	AC d_1 (in.)	Base d_2 (in.)	% of AC to Total Thick. (in.)	Thickness Of Stabilized Subgrade d_3 (in.)	Back-calculated Coefficient, a_3 (in.)	[1]SN_{actual}	[2]$SNr_{equired}$	AC Thickness Without stabilization $d_{1nonstab}$ (in.)	AC Thickness With stabilization d_{1stab} (in.)	AC Reduced Thick. Δd_{1AC} (in.)	Unit Cost of AC Unit cost X Δd_{1AC} ($)/ yd^2	Cost of Stab. Unit cost X d_3 ($)/ yd^2	Unit cost ($)/ yd^2	Savings (dollars) per mile x 36 ft width Flexoible Pavement
Anderson	3.2	2	13	4	76.5	12	0.00	6.28	6.3	13.0	10.2	2.9	5.41	4.23	1.18	24,837
Boyle	9.4	4	14	4	77.8	8	-0.01	6.72	6.57	13.7	11.7	1.9	3.60	2.82	0.78	16,558
Boone	0.65	5	8	4.5	64.0	11	-0.02 to-0.03	4.15	4.39	8.5	5.9	2.7	4.96	3.88	1.08	22,767
Breckinridge	1.9	4	11	5	68.8	10	0.00	5.54	5.3	10.5	7.6	2.9	5.40	4.90	0.50	10,507
Fayette	4.75	3	11	7	61.1	8.5	0.00	5.82	6.43	12.4	10.3	2.0	3.83	3.00	0.83	17,593
Daviess	1.0	6	8	0	100.0	22	0.04	3.52	4.35	9.9	3.5	6.4	11.87	10.78	1.09	23,116
Hardin 1	4.4	3	11	4.5	71.0	13	0.00	5.47	6.28	12.8	9.7	3.1	5.86	4.58	1.28	26,907
Hardin 2	*1.5*	*3*	*11*	*5*	*68.8*	*11*	*0.05 to0.10*	5.54	5.45	10.8	8.1	2.7	4.96	3.88	1.08	22,767
Hickman	1.3	5	9	6	60.0	10	0.00	4.8	4.85	9.1	6.6	2.5	4.68	3.00	1.68	35,455
Lee (AFBC)	*1.3*	*2*	*9*	*5*	*64.3*	*12*	*0.09*	*4.66*	*5.83*	11.7	9.5	2.2	4.08	3.60	0.48	10,233
Lee (10% Cement)	*1.3*	*2*	*7*	*5*	*58.3*	*12*	*0.18*	*3.78*	*5.9*	11.8	8.4	3.5	6.48	5.88	0.60	12,609
Lee (Lime)	*1.3*	*2*	*9*	*5*	*64.3*	*12*	*0.09*	*4.66*	*5.83*	11.7	8.8	2.9	5.41	4.23	1.18	24,837
Lee (Kiln Dust)	*1.3*	*2*	*9*	*5*	*64.3*	*12*	*0.10*	*4.66*	*5.83*	11.7	8.9	2.7	5.10	3.60	1.50	31,775
Lee (7% Cement)	*1.3*	*2*	*7.5*	*5*	*60.0*	*12*	*0.16*	*4*	*5.88*	11.8	8.3	3.5	6.48	5.88	0.60	12,609
Lee (AFBC)	*1.3*	*2*	*7.5*	*5*	*60.0*	*12*	*0.15*	*4*	*5.88*	11.8	9.6	2.2	4.08	3.60	0.48	10,233
McCracken	2.0	11	8	4	66.7	12	0.01	4.08	4.2	8.3	5.4	2.9	5.41	4.23	1.18	24,837
McCreary	3.3	6	7.5	5	60.0	10	0.10	4	5.31	10.5	8.1	2.4	4.51	3.53	0.98	20,698
Owen	0.6	2	9.5	4	70.4	7.5	0.03	4.74	5.19	10.5	8.7	1.8	3.38	2.64	0.74	15,523
Shelby	*2.4*	*2*	*11*	*0*	*100.0*	*8*	*0.19*	*4.84*	*5.96*	13.5	11.6	1.9	3.60	2.82	0.78	16,558
Trigg	3.4	2	9.5	5	65.5	11	0.00	4.88	6.55	13.3	10.6	2.7	4.96	3.88	1.08	22,767
Lee (None)[1]	*1.3*	*2*	*10*	*6*	*62.5*	*0*	*0.05*	5.24	5.85	11.4	11.4	0.0	0.00	0.00	0.00	0
McCracken[1]	2.0	11	8	4	66.7	0	0.01	4.08	4.2	8.3	8.3	0.0	0.00	0.00	0.00	0

1. *Subgrade was not stabilized.*

SUMMARY AND CONCLUSIONS

The long-term durability and performances of 20 flexible pavement sections constructed on soil subgrades treated with chemical admixtures were examined. More than 400 core holes were drilled in the sections to perform in situ CBR tests, obtain undisturbed samples for laboratory testing, measure thicknesses of the pavement components of each section, and perform standard penetration tests. Also, Falling weight deflectometer (FWD) tests were performed on each section. Based on the test results and analysis, the following conclusions, comments, and observations are made:

1. Based on a survey, 26 states of 38 states responding to the survey used chemical admixtures to improve the bearing strengths of soil subgrades. All respondents noted that chemical stabilization was very beneficial. The most frequently used chemical admixtures were hydrated lime and Portland cement.
2. Mixing soils with chemical admixtures, such as hydrated lime, cement, or hydrated lime-based byproducts, significantly reduces the clay fraction (0.002-mm size) of soils. Clayey soils (CL and CH) generally are transformed to silts (ML) and sandy silts (SM) when treated. Reduction in the clay fraction (% finer than 0.002 mm-particle size), of soils improves engineering properties. Bearing strengths and shear strengths increase.
3. Field measurements showed that in situ CBR values of soil subgrades stabilized with different chemical admixtures were much greater than in situ CBR values of untreated soil subgrades. At the 85^{th} percentile test value, in situ CBR values of hydrated lime-soil, Portland cement-soil, hydrated lime/cement-soil, and LKD-soil subgrades were 27, 59, 32, and 24, respectively. The in situ CBR value of the untreated subgrade at the 85^{th} percentile test value was only 2. In situ CBR values at the 85^{th} percentile test value of the soil subgrades treated with chemical admixtures were approximately 12 to 30 times greater than the in situ CBR of the untreated soil subgrade. Below the 85^{th} percentile test value, the in situ CBR values of the treated subgrades were much greater than the untreated subgrade.
4. Layer coefficients, a_3, of hydrated lime-soil, cement-soil, hydrated lime/cement-soil, LKD-soil, and AFBC-soil were determined and proposed. Using the AASHTO relationship of a_3 and CBR and the CBR values of the stabilized subgrades at the 85^{th} percentile test value, proposed design values are 0.106, 0.127, 0.11, 0.10, and 0.08, respectively. Based on the CBR value of the untreated subgrade soils and the design assumption at the 85^{th} percentile test value, the layer coefficients were only 0.38 and 0.026, respectively.
5. At 11 study sections, no structural credit was given to the chemically stabilized subgrade. Using the 1981 Kentucky Design Curves, back-calculated values of the layer coefficient, a_3, ranged from about minus 0.03 to plus 0.03. At two other sites, the values were 0.04 and 0.05—structurally, small credit was given. A one site, the layer coefficient ranged from 0.05 to 0.10—some credit was given. At eight sites, structural credit had been given to the treated subgrades and the "in service" layer coefficients values ranged from 0.09 to 0.19. Pavements at those sites have performed very well. Based on the good performances of pavements where the in-service layer coefficients, a_3 (back calculated), ranged from 0.09 to 0.19, the design layer coefficients proposed above appear very reasonable.
6. Moisture content data show that a soft layer of soil frequently exists at the top of untreated subgrades. On the basis of percentile test value, moisture contents measured at the very top of untreated subgrades were some 3-4 percent larger than moisture contents measured at points below

the top of the subgrades. This is a significant finding and has major engineering implications. By using chemical subgrade stabilization, the effects of the " soft zone" on pavements are eliminated, or mitigated, because the soft zone is positioned at a lower level in the subgrade where traffic stresses, and the effects of traffic stresses, are much less.

7. Resilient modulus values of soil-cement subgrades were much larger than values of resilient modulus of the unsaturated, non-stabilized (untreated) subgrades. Values of resilient modulus of the soil-cement subgrades were about 9,000 to 90,000 psi larger than resilient modulus values of the unsaturated, untreated subgrades. Resilient modulus values of soil-hydrated lime subgrades were about the same as values of resilient modulus of unsaturated, untreated subgrades. However below the 20th percentile test value, the resilient modulus values of the soil-hydrated lime subgrades were much larger than values of resilient modulus of the unsaturated, untreated subgrade. Based on laboratory tests, resilient moduli of saturated, untreated specimens are much lower than values of resilient modulus of unsaturated, untreated specimens, soil-cement specimens, and soil-hydrated lime specimens.

8. Average values of (back-calculated) modulus, determined from falling weight deflectometer measurements, of chemically stabilized subgrades were much larger than FWD values of modulus of the (unsaturated) untreated subgrades. Modulus values of the chemically stabilized subgrades ranged from 21,600 to 130,000 psi while the modulus values of the untreated subgrades ranged from 2,700 to 66,100 psi.

9. As the stiffness of the chemically stabilized subgrade increases, FWD modulus of the granular base increases. Average FWD back-calculated values of modulus of base aggregates –resting on the chemically stabilized subgrades—were larger than values of modulus of the stabilized subgrades. However, the FWD modulus of an aggregate base, resting on a stiff, treated subgrade layer, increase as the modulus of the chemically treated subgrade increase. For instance, as the modulus of soil-cement subgrades increases from about 27,000 to 100,000 psi, the modulus of the base aggregates increases from 19,630 to 220,000 psi. As the modulus of the soil-hydrated lime subgrades increases from 27,000 to 100,000 psi, the modulus of the base aggregates increases from 19,630 to 140,000 psi. When the modulus values of the soil-cement and soil-hydrated lime were identical, or equal to 27,000 psi, the modulus of the base aggregate was a constant and equal to 19,630 psi. The approximate value of 19,600 psi may represent a "thresh-hold" value of modulus. Obviously, modulus values of base aggregates resting on untreated subgrades (especially soft and saturated subgrades) will be much lower than modulus values of base aggregates resting on chemically treated subgrades. Evaluations of FWD modulus of base aggregates resting on untreated soil subgrade need further study.

10. Increasing the modulus of the base aggregate is major benefit of chemical stabilization. For instance, the layer coefficient, a_2, of granular base is generally accepted to be about 0.14 at a modulus value of about 30,000 psi. If the base modulus increases, than the layer coefficient increases. For example, if the base aggregate increases from 30,000 to 60,000, then the layer coefficient increases from 0.14 to 0.26. Since chemical stabilization of the subgrade increases the modulus of base aggregate, the layer coefficient of the base aggregate increases. If the modulus of the base aggregate increases, then the structural number of the pavement increases. Consequently, the overall structural integrity of the pavement structure is improved when chemical subgrade stabilization is used.

11. At the 50th and 20th percentile test values, average rutting values for the sites where measurements could be obtained ranged from 0.11 to 0.29 inches and 0.16 to 0.31 inches respectively. Averages for those percentile test values were 0.20 and 0.27, respectively. Rutting

values of the sections were reasonable small, considering that the ages of the sections ranged from about 7 to 15 years.

12. Chemical stabilization represents a very economical means of improving the poor engineering strengths of Kentucky soils. Moreover, the thickness of a pavement resting on a treated subgrade can be thinner than the thickness of a pavement resting on an untreated subgrade. For two pavement sections with equivalent structural numbers, SN, the cost of a pavement section resting on an untreated subgrade is greater than the cost of a pavement resting on a treated subgrade.

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APPENDIX A

Survey on the Usage of Chemical Admixtures in the United States for Stabilizing Highway Subgrades

October 20, 1998

State Highway Department
XXXXXXXXXXXXXXX
XXXXXXXXXXXXXXX

Dear ______?

The Kentucky Transportation Center is investigating the long-term benefits of highway subgrade stabilization methods utilized by the Kentucky Transportation Cabinet, Department of Highways. We are conducting a survey of state DOT=s to determine types of subgrade stabilization used and if subgrade stabilization is beneficial.

Please complete the enclosed survey and or forward results to the Kentucky Transportation Center.

Fell free to include any comments or information such as percentage of stabilizer, testing or construction standard reference that you think is useful.

If your agency is not the appropriate unit, please forward this inquiry to a unit familiar with subgrade design and construction.

Thank you for your assistance.

1. Is subgrade stabilization used in your state? Yes _____ No

If yes then:

2. What criterion is used to determine when subgrade stabilization will be performed? (Low strength subgrade soil, high traffic ESAL-s, etc.)

3. What type of stabilization is used?

Chemical

- ❑ Hydrated lime
- ❑ Types of soil stabilized with hydrated lime (e.g., PI > 20, Fat clay)
- ❑ Quick lime
- ❑ Types of soil stabilized with quick lime (e.g., PI > 20, Fat clay)
- ❑ Portland cement
- ❑ Types of soil stabilized with Portland cement (e.g., PI< 20, silts, sands)
- ❑ Industrial Byproducts (kiln dust, fly ash, etc)
- ❑ Asphalt
- ❑ Other
- ● Comments:

Mechanical

- ❑ Proof rolling
- ❑ Compaction
- ❑ Compaction Specification (example: 95% of standard proctor, ± 2% optimum moisture content)
- ❑ Geogrids
- ❑ Geofabrics
- ❑ Crushed stone
- ❑ Geofabrics or geogrids and crushed stone
- ❑ Other
- ● Comments:

4. Is the stabilized subgrade given structural credit in pavement design? Yes _____ No
- ● Comments:

5. Do you feel subgrade stabilization is beneficial? Yes _____ No
- ● Comments:

6. May we contact you in the future? Yes _____ No

- ● Telephone or e-mail address:

Table A-1. Survey results of the usage of highway chemical subgrade stabilization in the United States.

State	Stabili-zation Used	Criterion	Type Chemical	 Mechanical	Structural Credit	Beneficial
Alabama	Yes	Low CBR High PI	Hydrated and quick lime: CBR < 5 PI > 12	Compaction 100 % T-99 ± 2% omc Crushed Stone	Yes: Improved Roadbed (select material): 0.05 Stabilized Roadbed local or commercial material: 0.05 Lime Stabilized Roadbed : 0.10	Yes: Provides a uniform construction platform and foundation for pavement structure
Arizona	Yes	R-value <15	Hydrated lime: Clay Portland cement: sandy, silty soil	Geogrids Geofabrics	Yes: Use of stabilization, geogrid, or geofabric adds 10 points to subgrade R-value	Yes
Arkansas	Yes	Low strength soils or wet subgrade	Hydrate and quick lime: PI > 20 Portland cement: PI< 12	Proof rolling Compaction 95 % T-99 Geogrids Crushed stone or clean gravel to bridge soft areas	Yes: Structural Number of 0.07/in. And 0.20 /in. coefficient or relative strength per inch of treated depth for lime stabilized and Portland cement treated subgrade, respectively	Yes
California	Yes	Clay soil, R-value <10 Expansive Soil Low strength subgrade soil	Hydrated lime: Fat clays R-Value <10 Quick lime: Fat clays R-Value <10; used less frequently	Proof rolling: Compaction: 95 % of Caltrans compaction test, ≈T-180 Geogrids, geofabrics, crushed stone geogrids/geofabrics with crushed stone used sometimes	Yes: stabilized subgrade is considered to have properties of an aggregate subbase	Yes: in wet clay
Connecticut	Yes	Weak subgrade soils			Excavate and replace with suitable material Geogrids & geofabrics used occasionally Geogrids & geofabrics used with crushed stone on granular subbase	No

Table A-2. Survey results of the usage of highway chemical subgrade stabilization in the United States.

Florida	Yes	When Lime rock Bearing Ratio (LBR) of subgrade < 40, (CBR =32), subgrade stabilization is required	Chemical stabilization seldom used		98% modified proctor (T-180) Lime rock or clayey spoils Florida has an abundance of lime rock which makes it a cost effective stabilizer	Yes: stabilized subgrade is given a coefficient of 0.08
Idaho	Yes	Low subgrade strength particularly with high ESALs and pavement thickness constraints	Hydrated and quick lime: Fat clays Asphalt membrane over some fat clays: marginal results	Proof rolling: occasionally Compaction; 95 - 100% of T-99 depending on soil type Geogrids; occasionally used to reduce pavement thickness Geofabrics: routinely used as subgrade separator Crushed Stone: minus 3",clean shot rock, as a drainable base 12" thick Granular borrow: used as subgrade improvement SE > 30	Yes	Yes: For most part, some installations have not worked out, usually a construction problem more than treatment related
Illinois	Yes	Mechanistic Pavement Design, based on resilient modulus (as a function of grain size)	Hydrated lime: Minimum of 15 - 20% clay Portland cement: sands & silts Lime kiln dust - fly ash being evaluated	Proof rolling Compaction Geogrids and crushed stone	Yes: Only in stabilization, Not given structural credit in "modification", when subgrade is modified to provide a temporary construction platform	Yes: long-term benefits were not achieved as evidenced from some field observations
Indiana	Yes	Low strength high traffic such as interstate	Portland cement: non plastic silts and sands Lime, Lime kiln dust, & fly ash: used for drying wet subgrades but no strength is accounted for	Proof rolling Compaction: 95% T-99, -2 - + 1% omc Geogrids: subgrade modification Geofabrics: used with under drains, under rip rap Crushed Stone: used in subgrade modification	Yes: Soil stabilization and crushed stone are accounted for in raising the strength of soil	Yes: Saves time open road faster

Table A-3. Survey results of the usage of highway chemical subgrade stabilization in the United States.

Iowa	Yes	In design; When on-site "select" soils are not available for subgrade treatment, use Special backfill (granular material) with or without geogrid. During construction: Occasionally use/allow fly ash to stabilize soft/wet areas.	Hydrated lime: Years ago, but not currently Fly ash: typically in recent applications Asphalt: Years ago, but not currently	Proof rolling: Prior to paving Compaction: standard compaction (T-99) Geogrids with granular backfill	No: Not the "subgrade treatment" portion of stabilized subgrade	Yes
Kansas	Yes	Swell potential > 2% Construction expediency during reconstruction to prevent delays due to wet subgrades.	Hydrated and quick lime: Clays with > 2% swell Portland cement: PI < 8 Fly ash Type C: PI > 8 < 25	Compacting Type AA or B @ MR 5 Crushed stone: subgrade modification of low PI soil	Yes: Lime stabilized subgrade has an AASHTO structural coefficient of 0.11	Yes: Provide all weather working platform. Increased performance life of pavement. Cost effective for reducing paving materials Promotes reconstruction
Kentucky	Yes	Fine-grained soil when 85 % of CBR values < 6	Quick Lime- Clay Portland Cement- Silt Kiln Dust- Silt	Compaction 95% T-99 remove and replace soft soil with crushed stone and geogrids/geofabric	Limited	Yes Stabilization program began in late 1980's
Louisiana	Yes	All soils under Class I base (High Traffic) low strength subgrades under Class II Base specified as "treated" instead of "stabilized"	Hydrated and quick lime: sand ≤79%, or silt≤ 69% & PI≤ 35 Portland cement PI≤20 lime & cement when PI 21 - 35	Compaction: soil satisfaction of engineer Aggregate Subgrade layer 95% T-180 geogrids/geofabrics when specified crushed stone geogrids/geofabrics with crushed stone when specified	Yes	Yes

Table A-4. Survey results of the usage of highway chemical subgrade stabilization in the United States.

Minnesota	Yes	Low Strength Soil	Hydrated lime: for drying wet soils Fly ash: 1 research project using fly ash	Proof rolling - on large projects, when roadbed is completed 2 wheels 1.8 m apart Tire 18 X 25 13.7 metric tons on each wheel Compaction-All Projects Upper 3 feet 100% T-99, 65 - 102 % omc Below 3 feet 95% T-99, <115% 0mc	No	Yes
Maine	Yes	Low Strength Subgrades	Portland cement: mixed with base material, some subgrades Asphalt: Emulsified $CaCl_2$: All used experimentally with varied results	Proof rolling: passes to make stable relative to natural condition Compaction: 90% T-180 Geogrids: limited use as research in subgrade and base Geofabrics:if specified Crushed stone: to replace wet or soft soils	No	Yes: Stabilization retar frost heaving
Maine	No			Compaction: 95% T-99, ± 2% omc Experimental Section of roadway using geogrids and geosynthetics constructed in 1997 and being evaluated		
Maryland	Yes	Low strength subgrade soil M_r <4,500 psi soils with history of construction/performa nce problems e.g./. Micaceous silts, uniform fine sands w/o fines	Hydrated and quick lime: PI > 20±, micaceous silts w/ PI Portland cement: Low Pi, NP micaceous silts Percentage of stabilizer determined by laboratory testing	Proof rolling required on all subgrades Compaction: 97% T-180, ± 2% omc for top foot of subgrade Crushed stone geofabrics/geogrids and crushed stone	Yes: may be given credit depending on project conditions	Yes: Construction/workir platform and improv long-term performance. Stabilization methoc must match soil conditions

Table A-5. Survey results of the usage of highway chemical subgrade stabilization in the United States.

Mississippi	Yes	A-6 & A-7 soils Subbase stabilization on all new projects subbase is 7 in. Of granular material between subgrade and pavement	Hydrated and quick lime: Subgrades 3 - 6% lime Granular subbase with lime-fly ash 4% lime 8 - 12% fly ash	95% T-99 Geogrids, geofabrics, crushed stone rarely used	Yes: Lime fly ash sub base only	Yes
Montana	Yes	Low strength and high moisture content		Proof rolloing Compaction Geogrids Geofabrics Crushed Stone Geofarics or Geogrids w/ crushed stone	Yes: Upper 2.0 feet is given structural credit if replaced by FHWA mandate	Yes
Nebraska	Yes	Sandy and wet soils	Fly ash: for wet slit-clays Soil binder for sandy soils	> 100 % T-180	No	Yes
New Hampshire	No			2' - 4' of select granular material over subgrade Lot of HMA and PCC reclaiming and pulverizing		
Michigan	Yes	Regional soils engineer responsible for soils assessment		Proof rolling Compaction: 95 % T-99, @ max +2% omc below top 1 meter 95% T-99 @ max 0% for top 1 meter Geogrids w/sand; sometimes stone or blast furnace slag for lightweight fill Geofabrics w/sand backfill Expanded polystyrene and foamed concrete as lightweight fill for site specific conditions in lieu of remove and replace or other subgrade stabilization	Yes: Increased M_r for flexible and “k’ with rigid pavements	Yes: stable subgrade is essential to maintaining integrity of base course

Table A-6. Survey results of the usage of highway chemical subgrade stabilization in the United States.

New York	No		Have used lime and soil-cement stabilization. However, no State projects stabilized in approximately 25 years. Have hydrated lime and soil cement specifications.			
North Carolina	Yes	Poor subgrade soils Type of facility Traffic control needs Volume of stabilization required	Hydrated lime slurry: PI > 10, silty and clayey soils Quick lime: Spot stabilization and rural projects, silty and clayey soils Portland cement: PI < 10, silty and sandy soils	Proof rolling: Compaction: 97 % T-99, ± 2% optimum moisture content Geogrids Geofabrics Crushed stone Geofabrics/geogrids and crushed stone	Yes: for lime and cement No: for mechanical stabilization, eg. Fabrics, crushed stone	Yes: Provides a stable working platform for paving operations Chemical stabilization reduces moisture susceptibility problems
North Dakota	Yes	Low Strength Soil FWD Pavement Distress # ESAL's		95 % T-99 -4 - +5% mc 85% T-180 0 - +5% mc Crushed Stone Fabrics Stone and fabrics Increasing use of fabrics	No	Yes: Working Platform Extend Pavement Life
Ohio	Yes	Aid in constructability due to weak or wet soils	Hydrated and quick lime: PI > 16 Added to standard specifications in 1997	Compaction: No soils less than 100 lbs/ft^3 (T-99) used in upper 12 inches of subgrade ≥102% T-99 if max dry density between 100 -105lbs/ft^3, ≥100% T-99 for all other soils Proof rolling on large jobs Geogrids/fabrics/crushed stone to remediate small areas	No	Yes
Pennsylvania	Yes	Poor subgrade conditions-weak, wet, unstable under compaction		Proof rolling: Used to determine if subgrade is stable Compaction: rework and recompact, 100% T-99 ±2% omc Geogrids Geofabrics Crushed stone Geogrids/geofabrics and crushed stone	No	Yes

Table A-7. Survey results of the usage of highway chemical subgrade stabilization in the United States.

Rhode Island	No					Yes: Currently investigating use of geosynthetics for stabilization of soft soil shoulders
South Carolina	Yes	Low strength subgrade soils SCDOT uses soil support values based on CBR tests	Portland cement: Normally clays Use of cement-modified subgrades has been successful state-wide with a variety of soil types	Compaction: 95% T-99 Geogrids & geofabrics used occasionally	Yes: Structural coefficient of 0.15 used	Yes
South Dakota	Yes			Compaction Specification Geogrids Geofabrics Crushed Stone Geogrids/fabrics/crushed stone		Yes
Tennessee	Yes	Low Strength Soil	Hydrated lime: A-7-6 & A-6 soil with low CBR (1 - 3) Portland cement- silts with low CBR (1 - 3)	Compaction: 100% T-99 top 6 inches 95% T-99 rest limited use of geogrids	Yes	Yes
Texas	Yes	Weak subgrade, High PI subgrades subjected to extreme wet dry cycles absence of cheaper alternate	Hydrated and quick lime: PI >20 Portland cement, PI < 20 Industrial Byproducts Asphalt	Limited use of geogrids and geofabrics	Yes if: Stabilization considered permanent passes freeze-thaw durability requirements No if: considered treatment no structural credit	Yes: We believe in building pavements from bottom up and pay special attention to subgrade as we will probably never see it again

Table A-8. Survey results of the usage of highway chemical subgrade stabilization in the United States.

Utah	Yes		Hydrated and quick lime: A-7-5 soils Portland cement: Non-Plastic soils Asphalt	Proof rolling Compaction Crushed stone	Yes	Yes
Vermont	No					
Virginia	Yes	Low CBR High In situ Moisture Contents	Hydrated and quick lime Portland cement Fly ash rarely used	Compaction: 100% T-99 ± 3% mc for top 150 mm Geogrids & geofabrics used to stabilize poor subgrades and embankment foundations Crushed Stone Used for removal and replacement of poor soils	Yes: 0.4 equivalency	Yes: Difficult to achieve aggregate base density in low CBR soils
West Virginia	No	Granular subgrade, which is a low quality base used to replaces unstable subgrade		Geogrids, Geofabrics, and crushed stone used in subbase	No: Subgrade is not stabilized and used in pavement design	Depends on type of material used for subgrade; natural soils or granular material
Wisconsin	Yes	Low strength subgrade Excess deformation during construction Mostly silt soils High moisture content	Hydrated lime: Limited use in clays Byproducts very limited use Lime and byproducts used primarily as drying agents	Proof rolling: new specification being developed Compaction: 95 % T-99 no moisture control Compaction: 95 % T-99 ≤ 110% omc	No	Yes
Wyoming	Yes		Hydrated lime: Used occasionally on reconstruction projects mostly wet silts	Compaction: 95 % T-99 -4 to +2% omc Geofabrics and crushed stone: Used in soft areas Cuts excavated to ditch bottom and compacted	No	Yes: Mechanical Chemical only a few beneficial circumstances

Appendix B

Global Positioning System (GPS) Locations of Pavement Borings Latitudes and Longitudes

Table B -1. GPS positions of Tested Sites

ID	Attributes	(recorded in datalogger)	File name (rover)	Workspace	Longitude (DD)	Latitude (DD	Elevation (HAE)
4	"""Borings"""	"""BOONE 842 10+00 0.2 AC ST1"""	"""R110115A.cor"""	"""geotech2"""	-84.61923856	38.96883804	251.238
5	"""Borings"""	"""BOONE 842 30+00 0.55 ST1"""	"""R110115A.cor"""	"""geotech2"""	-84.62560232	38.96731692	240.187
6	"""Borings"""	"""BOONE 842 30+00 0.55 ST2"""	"""R110115A.cor"""	"""geotech2"""	-84.62563069	38.96731103	240.694
7	"""Borings"""	"""BOONE 842 30+00 0.55 CBR"""	"""R110115A.cor"""	"""geotech2"""	-84.62569254	38.96728695	239.766
8	"""Borings"""	"""BOONE 842 30+00 0.55 SPT"""	"""R110115A.cor"""	"""geotech2"""	-84.62569992	38.96728564	240.07
9	"""Borings"""	"""BOONE 842 50+00 0.95 ST1"""	"""R110115A.cor"""	"""geotech2"""	-84.63231271	38.96552826	232.186
10	"""Borings"""	"""BOONE 842 50+00 0.95 TCORE ST2"""	"""R110115A.cor"""	"""geotech2"""	-84.6323281	38.96553115	232.128
11	"""Borings"""	"""BOONE 842 50+00 0.95 SPT"""	"""R110115A.cor"""	"""geotech2"""	-84.63242482	38.96552846	232.664
12	"""Borings"""	"""BOONE 842 50+00 0.95 CBR"""	"""R110115A.cor"""	"""geotech2"""	-84.63245482	38.96553006	232.89
13	"""Borings"""	"""BOONE 842 70+00 1.35 CBR"""	"""R110115A.cor"""	"""geotech2"""	-84.63919019	38.96692783	239.188
14	"""Borings"""	"""BOONE 842 70+00 1.35 TCORE"""	"""R110115A.cor"""	"""geotech2"""	-84.63929303	38.96696058	238.444
15	"""Borings"""	"""BOONE 842 70+00 1.35 ST1"""	"""R110115A.cor"""	"""geotech2"""	-84.63917189	38.96692688	238.895
16	"""Borings"""	"""BOONE 842 70+00 1.35 SPT"""	"""R110115A.cor"""	"""geotech2"""	-84.63915605	38.96692465	239.593
17	"""Borings"""	"""BOONE 842 90+00 1.70 ST1"""	"""R110115A.cor"""	"""geotech2"""	-84.64525689	38.96932545	239.13
18	"""Borings"""	"""BOONE 842 90+00 1.70 SPT"""	"""R110115A.cor"""	"""geotech2"""	-84.6452626	38.96931953	236.616
19	"""Borings"""	"""BOONE 842 90+00 1.70 CBR"""	"""R110115A.cor"""	"""geotech2"""	-84.64520543	38.9692928	237.086
20	"""Borings"""	"""BOONE 842 90+00 1.70 ST2"""	"""R110115A.cor"""	"""geotech2"""	-84.64513408	38.96923585	235.617
21	"""Borings"""	"""BOONE 842 110+00 2.1 AC ST1"""	"""R110115A.cor"""	"""geotech2"""	-84.65067106	38.9727838	229.419
22	"""Borings"""	"""BOONE 842 110+00 2.1 CBR"""	"""R110115A.cor"""	"""geotech2"""	-84.6507296	38.9728198	229.864
23	"""Borings"""	"""BOONE 842 110+00 2.1 SPT"""	"""R110115A.cor"""	"""geotech2"""	-84.65075627	38.97283381	229.228
24	"""Borings"""	"""BOONE 842 120+00 2.3 SPT"""	"""R110115A.cor"""	"""geotech2"""	-84.65349822	38.97455069	227.752
27	"""Borings"""	"""ShelbyKy 55 NB 10+00 HOLE 1 CBR."""	"""ky55.cor"""	"""geotech2"""	-85.20425623	38.21344341	190.879
28	"""Borings"""	"""KY55 NORTH 10+00 HOLE 2 SUB CR"""	"""ky55.cor"""	"""geotech2"""	-85.20426217	38.21346279	191.643
29	"""Borings"""	"""KY55 NORTH 10+00 HOLE 3 ST. C"""	"""ky55.cor"""	"""geotech2"""	-85.2042616	38.21348786	191.188
30	"""Borings"""	"""KY55 NORTH 10+00 HOLE 4 SPT. C"""	"""ky55.cor"""	"""geotech2"""	-85.20426083	38.21350839	191.243
31	"""Borings"""	"""KY55 NORTH 30+00 HOLE 1 SPT."""	"""ky55.cor"""	"""geotech2"""	-85.20344442	38.21906346	199.253
32	"""Borings"""	"""KY55 NORTH 30+00 HOLE 2 ST2."""	"""ky55.cor"""	"""geotech2"""	-85.20343694	38.21907825	200.261
33	"""Borings"""	"""KY55 NORTH 30+00 HOLE 3 CBR"""	"""ky55.cor"""	"""geotech2"""	-85.20343653	38.21909244	200.683
34	"""Borings"""	"""KY55 NORTH 30+00 HOLE 4 ST1"""	"""ky55.cor"""	"""geotech2"""	-85.20343493	38.21910637	200.339
35	"""Borings"""	"""KY55 NORTH 50+00 HOLE 1 ST1CLG"""	"""ky55.cor"""	"""geotech2"""	-85.20264121	38.22429632	195.25
36	"""Borings"""	"""KY55 NORTH 50+00 HOLE 2 CBR"""	"""ky55.cor"""	"""geotech2"""	-85.2026383	38.22432993	195.267
37	"""Borings"""	"""KY55 NORTH 50+00 HOLE 3 ST"""	"""ky55.cor"""	"""geotech2"""	-85.20263454	38.22435366	194.47
38	"""Borings"""	"""KY55 NORTH 50+00 HOLE 4 SPT"""	"""ky55.cor"""	"""geotech2"""	-85.20263545	38.22437389	194.4
40	"""Borings"""	"""KY55 SOUTH 60+00 HOLE ST1"""	"""ky55.cor"""	"""geotech2"""	-85.20247334	38.22709045	191.551
41	"""Borings"""	"""KY55 SOUTH 60+00 HOLE 2 ST2"""	"""ky55.cor"""	"""geotech2"""	-85.20247981	38.227076	192.329
42	"""Borings"""	"""KY55 SOUTH 60+00 HOLE 3 SPT"""	"""ky55.cor"""	"""geotech2"""	-85.20248594	38.22706651	190.353
43	"""Borings"""	"""KY55 SOUTH 60+00 HOLE 4 CBR"""	"""ky55.cor"""	"""geotech2"""	-85.20248705	38.22700479	191.322
44	"""Borings"""	"""KY55 SOUTH 40+00 HOLE 1 ST1"""	"""ky55.cor"""	"""geotech2"""	-85.2030272	38.22167457	198.144
45	"""Borings"""	"""KY55 SOUTH 40+00 HOLE 2 SPT"""	"""ky55.cor"""	"""geotech2"""	-85.20303826	38.22160727	197.489
46	"""Borings"""	"""KY55 SOUTH 40+00 HOLE 3 ST2"""	"""ky55.cor"""	"""geotech2"""	-85.20304174	38.22158796	198.146
47	"""Borings"""	"""KY55 SOUTH 40+00 HOLE 4 CBR"""	"""ky55.cor"""	"""geotech2"""	-85.20304731	38.22154234	197.676
48	"""Borings"""	"""KY55 SOUTH 20+00 HOLE 1 ST1"""	"""ky55.cor"""	"""geotech2"""	-85.20399501	38.21624529	197.875
49	"""Borings"""	"""KY55 SOUTH 20+00 HOLE 2 SPT"""	"""ky55.cor"""	`"""geotech2"""	-85.204	38.21622447	198.472
50	"""Borings"""	"""KY55 SOUTH 20+00 HOLE 3 ST2"""	"""ky55.cor"""	"""geotech2"""	-85.2040062	38.21619706	199.338
51	"""Borings"""	"""KY55 SOUTH 20+00 HOLE 4 CBR"""	"""ky55.cor"""	"""geotech2"""	-85.20401407	38.21614382	198.368

Table B-2. GPS positions of Tested Sites

ID	Attributes	(recorded in datalogger)	File name (rover)	Workspace	Longitude (DD)	Latitude (DD	Elevation (HAE)
52	"""Borings"""	""" US 25 STA 10+00 SPT"""	"""R042213A.cor"""	"""geotech2"""	-84.51042611	38.08535174	261.964
53	"""Borings"""	"""US 25 STA 10+00 st1"""	"""R042213A.cor"""	"""geotech2"""	-84.51042111	38.08537536	260.458
54	"""Borings"""	"""US 25 STA 10+00 st2"""	"""R042213A.cor"""	"""geotech2"""	-84.51044502	38.08539069	260.734
55	"""Borings"""	"""US 25 STA 10+00 cbr"""	"""R042213A.cor"""	"""geotech2"""	-84.51050244	38.08553926	261.907
56	"""Borings"""	"""US 25 STA 30+00 st"""	"""R042213A.cor"""	"""geotech2"""	-84.51181127	38.09100298	242.525
57	"""Borings"""	"""US 25 STA 30+00 st"""	"""R042213A.cor"""	"""geotech2"""	-84.51204917	38.09083306	287.692
58	"""Borings"""	"""US 25 STA 30+00 spt"""	"""R042213A.cor"""	"""geotech2"""	-84.51209312	38.09100554	260.3
59	"""Borings"""	"""US 25 STA 50+00 cbr"""	"""R042213A.cor"""	"""geotech2"""	-84.51319377	38.09623446	261.713
60	"""Borings"""	"""US 25 STA 50+00 st"""	"""R042213A.cor"""	"""geotech2"""	-84.51325058	38.09632042	255.873
61	"""Borings"""	"""US 25 STA 50+00 spt"""	"""R042213A.cor"""	"""geotech2"""	-84.51324815	38.09632691	255.058
62	"""Borings"""	"""US 25 STA 70+00 spt"""	"""R042213A.cor"""	"""geotech2"""	-84.51438455	38.10189931	267.814
63	"""Borings"""	"""US 25 STA 70+00 st1"""	"""R042213A.cor"""	"""geotech2"""	-84.51417593	38.10172686	251.827
64	"""Borings"""	"""US 25 STA 70+00 st2"""	"""R042213A.cor"""	"""geotech2"""	-84.51424554	38.10185768	253.491
65	"""Borings"""	"""US 25 STA 70+00 cbr"""	"""R042213A.cor"""	"""geotech2"""	-84.51423176	38.10181143	253.999
66	"""Borings"""	"""US 25 STA 81+00 spt"""	"""R042213A.cor"""	"""geotech2"""	-84.51474803	38.10523455	167.517
67	"""Borings"""	"""US 25 STA 81+00 cbr"""	"""R042213A.cor"""	"""geotech2"""	-84.51510122	38.10480778	261.272
68	"""Borings"""	"""US 25 STA 81+00 st"""	"""R042213A.cor"""	"""geotech2"""	-84.51510137	38.10482907	259.271
69	"""Borings"""	"""US 25 STA 81+00 core"""	"""R042213A.cor"""	"""geotech2"""	-84.51514245	38.10489552	259.518
70	"""Borings"""	"""US 25 STA10+000 core"""	"""R042213A.cor"""	"""geotech2"""	-84.51773626	38.10927595	250.576
71	"""Borings"""	"""US 25 STA 77+75 st"""	"""R042213A.cor"""	"""geotech2"""	-84.51493085	38.10378613	262.881
72	"""Borings"""	"""US 25 STA 77+75 cbr"""	"""R042213A.cor"""	"""geotech2"""	-84.51490401	38.10367485	255.664
73	"""Borings"""	"""US 25 STA 77+75 spt"""	"""R042213A.cor"""	"""geotech2"""	-84.51485821	38.10368269	256.77
74	"""Borings"""	"""US 25 STA 60+00 st1"""	"""R042213A.cor"""	"""geotech2"""	-84.51384691	38.09937569	233.41
75	"""Borings"""	"""US 25 STA 60+00 st2"""	"""R042213A.cor"""	"""geotech2"""	-84.51391414	38.09909502	244.523
76	"""Borings"""	"""US 25 STA 60+00 spt"""	"""R042213A.cor"""	"""geotech2"""	-84.51389092	38.0989562	249.132
77	"""Borings"""	"""US 25 STA 60+00 cbr"""	"""R042213A.cor"""	"""geotech2"""	-84.513894	38.09893396	251.906
78	"""Borings"""	"""US 25 STA 40+00 spt"""	"""R042213A.cor"""	"""geotech2"""	-84.51299676	38.09388598	251.229
79	"""Borings"""	"""US 25 STA 40+00 cbr"""	"""R042213A.cor"""	"""geotech2"""	-84.51297259	38.09386205	261.718
80	"""Borings"""	"""US 25 STA 40+00 st1"""	"""R042213A.cor"""	"""geotech2"""	-84.5129576	38.0938148	262.914
81	"""Borings"""	"""US 25 STA 40+00 st2"""	"""R042213A.cor"""	"""geotech2"""	-84.51295541	38.09380318	264.348
82	"""Borings"""	"""US 25 STA 20+00 st1"""	"""R042213A.cor"""	"""geotech2"""	-84.5114578	38.08803263	263.553
83	"""Borings"""	"""US 25 STA 20+00 st2"""	"""R042213A.cor"""	"""geotech2"""	-84.51155033	38.08824662	260.893
84	"""Borings"""	"""US 25 STA 20+00 spt"""	"""R042213A.cor"""	"""geotech2"""	-84.51152103	38.08815561	259.902
85	"""Borings"""	"""US 25 STA 20+00 cbr"""	"""R042213A.cor"""	"""geotech2"""	-84.51151165	38.088127	258.789
86	"""Borings"""	"""US 25 STA -04+00 core"""	"""R042213A.cor"""	"""geotech2"""	-84.50914281	38.08183871	255.217
87	"""Borings"""	"""ANDERSON US 127 98+20 CBR"""	"""R060814a.cor"""	"""geotech2"""	-84.91982289	38.09465302	198.641
88	"""Borings"""	"""ANDERSON US 127 98+20 SPT"""	"""R060814a.cor"""	"""geotech2"""	-84.91982016	38.09456503	199.452
89	"""Borings"""	"""ANDERSON US 127 75+38 ST2"""	"""R060814a.cor"""	"""geotech2"""	-84.91902984	38.08843589	211.295
90	"""Borings"""	"""ANDERSON US 127 75+38 CBR"""	"""R060814a.cor"""	"""geotech2"""	-84.91902737	38.08839649	211.244
91	"""Borings"""	"""ANDERSON US 127 75+38 SPT"""	"""R060814a.cor"""	"""geotech2"""	-84.91901572	38.08835037	210.872
92	"""Borings"""	"""ANDERSON US 127 75+38 ST1"""	"""R060814a.cor"""	"""geotech2"""	-84.91901604	38.0883237	211.196
93	"""Borings"""	"""ANDERSON US 127 65+64 CORE"""	"""R060814a.cor"""	"""geotech2"""	-84.91867916	38.08591922	215.62
94	"""Borings"""	"""ANDERSON US 127 65+64 SPT"""	"""R060814a.cor"""	"""geotech2"""	-84.91866259	38.08580016	215.546
95	"""Borings"""	"""ANDERSON US 127 65+64 CBR"""	"""R060814a.cor"""	"""geotech2"""	-84.91865751	38.0857725	214.952
96	"""Borings"""	"""ANDERSON US 127 50+00 CBR"""	"""R060814a.cor"""	"""geotech2"""	-84.91807302	38.08160231	208.269
97	"""Borings"""	"""ANDERSON US 127 50+00 SPT"""	"""R060814a.cor"""	"""geotech2"""	-84.91807469	38.08162871	208.181

Table B-3. GPS positions of Tested Sites

ID	Attributes	(recorded in datalogger)	File name (rover)	Workspace	Longitude (DD)	Latitude (DD	Elevation (HAE)
98	"""Borings"""	"""ANDERSON US 127 50+00 SUBCORE"""	"""R060814a.cor"""	"""geotech2"""	-84.91805955	38.08150764	208.73
99	"""Borings"""	"""ANDERSON US 127 30+00 SPT"""	"""R060814a.cor"""	"""geotech2"""	-84.91727898	38.07607397	204.157
100	"""Borings"""	"""ANDERSON US 127 30+00 CBR"""	"""R060814a.cor"""	"""geotech2"""	-84.91727698	38.07605485	204.208
101	"""Borings"""	"""ANDERSON US 127 30+00 ST1 ST2"""	"""R060814a.cor"""	"""geotech2"""	-84.91727529	38.07603243	203.611
102	"""Borings"""	"""ANDERSON US 127 30+00 ST3"""	"""R060814a.cor"""	"""geotech2"""	-84.91727149	38.07600587	203.513
103	"""Borings"""	"""ANDERSON US 127 11+50 UK"""	"""R060814a.cor"""	"""geotech2"""	-84.91826463	38.07112068	204.159
104	"""Borings"""	"""ANDERSON US 127 11+50 UK"""	"""R060814a.cor"""	"""geotech2"""	-84.91826906	38.07111037	205.6
105	"""Borings"""	"""ANDERSON US 127 11+50 UK"""	"""R060814a.cor"""	"""geotech2"""	-84.91827388	38.0711058	206.409
106	"""Borings"""	"""LEE KY 11 9.9 SPT"""	"""R062415A.cor"""	"""geotech2"""	-83.70233617	37.64546943	313.483
107	"""Borings"""	"""LEE KY 11 9.9 CBR"""	"""R062415A.cor"""	"""geotech2"""	-83.70234051	37.64548131	314.372
108	"""Borings"""	"""LEE KY 11 9.9 ST1"""	"""R062415A.cor"""	"""geotech2"""	-83.7023498	37.64549089	314.09
109	"""Borings"""	"""LEE KY 11 9.9 AC CORE"""	"""R062415A.cor"""	"""geotech2"""	-83.70240589	37.64554731	316.256
110	"""Borings"""	"""LEE KY 11 10.15 SPT"""	"""R062415A.cor"""	"""geotech2"""	-83.70306568	37.64804022	322.542
111	"""Borings"""	"""LEE KY 11 10.15 CBR"""	"""R062415A.cor"""	"""geotech2"""	-83.70306585	37.64806678	320.604
112	"""Borings"""	"""LEE KY 11 10.15 ST1"""	"""R062415A.cor"""	"""geotech2"""	-83.70306244	37.64808049	319.459
113	"""Borings"""	"""LEE KY 11 10.15 ST2"""	"""R062415A.cor"""	"""geotech2"""	-83.70306099	37.64810476	319.578
114	"""Borings"""	"""LEE KY 11 10.30 CORETUBE""	"""R062415A.cor"""	"""geotech2"""	-83.70286329	37.64977013	321.146
115	"""Borings"""	"""LEE KY 11 10.30 SPT"""	"""R062415A.cor"""	"""geotech2"""	-83.70284042	37.64988353	320.544
116	"""Borings"""	"""LEE KY 11 10.30 CBR"""	"""R062415A.cor"""	"""geotech2"""	-83.70283158	37.6499171	319.656
117	"""Borings"""	"""LEE KY 11 10.50 ST1"""	"""R062415A.cor"""	"""geotech2"""	-83.70250305	37.65228912	323.334
118	"""Borings"""	"""LEE KY 11 10.50 SPT"""	"""R062415A.cor"""	"""geotech2"""	-83.70249278	37.65238762	322.82
119	"""Borings"""	"""LEE KY 11 10.50 CBR"""	"""R062415A.cor"""	"""geotech2"""	-83.7024845	37.65241193	323.047
120	"""Borings"""	"""LEE KY 11 11.0 AC CORE """	"""R062415A.cor"""	"""geotech2"""	-83.69896996	37.65903494	310.192
121	"""Borings"""	"""LEE KY 11 11.0 SPT"""	"""R062415A.cor"""	"""geotech2"""	-83.69888014	37.65909039	312.079
122	"""Borings"""	"""LEE KY 11 11.0 TRT CBR"""	"""R062415A.cor"""	"""geotech2"""	-83.69885677	37.65910124	310.69
123	"""Borings"""	"""LEE KY 11 11.0 UNTRTCBR"""	"""R062415A.cor"""	"""geotech2"""	-83.6987115	37.65919426	314.155
124	"""Borings"""	"""LEE KY 11 11.2 SPT"""	"""R062415A.cor"""	"""geotech2"""	-83.69581758	37.66104113	305.185
125	"""Borings"""	"""LEE KY 11 11.2 CBR"""	"""R062415A.cor"""	"""geotech2"""	-83.69579368	37.66106118	304.559
126	"""Borings"""	"""LEE KY 11 11.2 ST1 CORE"""	"""R062415A.cor"""	"""geotech2"""	-83.69572056	37.66110994	305.284
127	"""Borings"""	"""LEE KY 11 12.0 SPT"""	"""R062415A.cor"""	"""geotech2"""	-83.68394976	37.66858012	315.662
128	"""Borings"""	"""LEE KY 11 12.0 CBR"""	"""R062415A.cor"""	"""geotech2"""	-83.68395127	37.66858749	317.048
129	"""Borings"""	"""LEE KY 11 12.0 COREST1 """	"""R062415A.cor"""	"""geotech2"""	-83.68387146	37.66863571	316.97
130	"""Borings"""	"""LEE KY 11 12.5 CBR"""	"""R062415A.cor"""	"""geotech2"""	-83.67837539	37.67264563	327.445
131	"""Borings"""	"""LEE KY 11 12.5 SPT"""	"""R062415A.cor"""	"""geotech2"""	-83.67837236	37.67265746	329.643
132	"""Borings"""	"""LEE KY 11 12.5 CORE ST1"""	"""R062415A.cor"""	"""geotech2"""	-83.67833196	37.67271959	328.992
133	"""Borings"""	"""LEE KY 11 13.7 CORE ST"""	"""R062415A.cor"""	"""geotech2"""	-83.68423258	37.6882744	318.084
134	"""Borings"""	"""LEE KY 11 13.7 SPT"""	"""R062415A.cor"""	"""geotech2"""	-83.68423728	37.68837285	319.534
135	"""Borings"""	"""LEE KY 11 13.7 CBR"""	"""R062415A.cor"""	"""geotech2"""	-83.68424158	37.68838899	319.963
136	"""Borings"""	"""LEE KY 11 14.1 ST"""	"""R062415A.cor"""	"""geotech2"""	-83.68537207	37.69533357	319.845
137	"""Borings"""	"""LEE KY 11 14.1 SPT"""	"""R062415A.cor"""	"""geotech2"""	-83.68534478	37.69542278	319.453
138	"""Borings"""	"""LEE KY 11 14.1 CBR"""	"""R062415A.cor"""	"""geotech2"""	-83.68534225	37.69543669	319.63
139	"""Borings"""	"""LEE KY 11 14.5 SPT"""	"""R062415A.cor"""	"""geotech2"""	-83.68327725	37.70084052	327.933
140	"""Borings"""	"""LEE KY 11 14.5 CBR"""	"""R062415A.cor"""	"""geotech2"""	-83.68326976	37.70083806	329.778
141	"""Borings"""	"""LEE KY 11 14.5 ST1"""	"""R062415A.cor"""	"""geotech2"""	-83.68324228	37.70091956	329.498
142	"""Borings"""	"""LEE KY 11 14.5 ST2"""	"""R062415A.cor"""	"""geotech2"""	-83.6832394	37.70092079	329.917
143	"""Borings"""	"""LEE KY 11 14.7 SPT"""	"""R062415A.cor"""	"""geotech2"""	-83.68236733	37.70349396	336.754
144	"""Borings"""	"""LEE KY 11 14.7 CBR"""	"""R062415A.cor"""	"""geotech2"""	-83.68236095	37.70349574	336.877
145	"""Borings"""	"""LEE KY 11 14.7 ST1"""	"""R062415A.cor"""	"""geotech2"""	-83.68233566	37.70357341	335.344
146	"""Borings"""	"""LEE KY 11 14.7 ST2"""	"""R062415A.cor"""	"""geotech2"""	-83.68232882	37.7035895	334.809
147	"""Borings"""	""" US 27STA 789+00 AC """	"""R072614A.cor"""	"""geotech2"""	-84.48537169	36.84615058	351.699
148	"""Borings"""	"""STA 789+00 SPT"""	"""R072614A.cor"""	"""geotech2"""	-84.48532375	36.84608237	353.41

Table B-4. GPS positions of Tested Sites

ID	Attributes	(recorded in datalogger)	File name (rover)	Workspace	Longitude (DD)	Latitude (DD	Elevation (HAE)
149	"""Borings"""	"""STA 789+00 CBR"""	"""R072614A.cor"""	"""geotech2"""	-84.48529907	36.84606175	351.986
150	"""Borings"""	"""STA 774+20 CBR"""	"""R072614A.cor"""	"""geotech2"""	-84.48240249	36.84281664	358.716
151	"""Borings"""	"""STA 774+20 SPT"""	"""R072614A.cor"""	"""geotech2"""	-84.4823933	36.84279646	359.725
152	"""Borings"""	"""STA 774+20 AC CORE ST-1"""	"""R072614A.cor"""	"""geotech2"""	-84.4823526	36.84271143	359.671
153	"""Borings"""	"""STA 756+30 AC CORE ST-1"""	"""R072614A.cor"""	"""geotech2"""	-84.4818764	36.8379211	360.931
154	"""Borings"""	"""STA 756+30 SPT"""	"""R072614A.cor"""	"""geotech2"""	-84.48185501	36.8380037	362.045
155	"""Borings"""	"""STA 756+30 CBR"""	"""R072614A.cor"""	"""geotech2"""	-84.48185193	36.83801759	362.056
156	"""Borings"""	"""SST 733+40 AC CORE ST-1"""	"""R072614A.cor"""	"""geotech2"""	-84.48322942	36.83168797	352.593
157	"""Borings"""	"""SST 733+40 SPT"""	"""R072614A.cor"""	"""geotech2"""	-84.48321033	36.8317817	352.647
158	"""Borings"""	"""SST 733+40 CBR"""	"""R072614A.cor"""	"""geotech2"""	-84.48320574	36.83180163	352.692
159	"""Borings"""	"""ST 679+00 CBR"""	"""R072614A.cor"""	"""geotech2"""	-84.48698207	36.81696272	342.229
160	"""Borings"""	"""ST 679+00 SPT"""	"""R072614A.cor"""	"""geotech2"""	-84.4869759	36.81699723	341.759
161	"""Borings"""	"""ST 679+00 AC CORE ST-1"""	"""R072614A.cor"""	"""geotech2"""	-84.48693736	36.81708262	339.386
162	"""Borings"""	"""ST 655+75 CORE ST-1"""	"""R072614A.cor"""	"""geotech2"""	-84.48804011	36.81077785	370.814
163	"""Borings"""	"""ST 655+75 SPT"""	"""R072614A.cor"""	"""geotech2"""	-84.48803884	36.8107965	370.308
164	"""Borings"""	"""ST 655+75 CBR"""	"""R072614A.cor"""	"""geotech2"""	-84.48803823	36.81081216	370.113
165	"""Borings"""	"""ST 655+75 AC"""	"""R072614A.cor"""	"""geotech2"""	-84.48803429	36.81071478	370.518
166	"""Borings"""	"""HICKMAN US 51 STA 2+25 CBR"""	"""R080517A.cor"""	"""geotech2"""	-89.00279098	36.75202752	72.747
167	"""Borings"""	"""HICKMAN US 51 STA 2+25 SPT"""	"""R080517A.cor"""	"""geotech2"""	-89.00278749	36.75201661	74.548
168	"""Borings"""	"""HICKMAN US 51 STA 2+25 ST-2"""	"""R080517A.cor"""	"""geotech2"""	-89.00277177	36.75195254	75.792
169	"""Borings"""	"""HICKMAN US 51 STA 2+25 ST-1 AC"""	"""R080517A.cor"""	"""geotech2"""	-89.00276535	36.7519373	73.991
170	"""Borings"""	"""HICKMAN US 51 STA 15+00 ST1CBR"""	"""R080517A.cor"""	"""geotech2"""	-89.00331699	36.75548673	77.804
171	"""Borings"""	"""HICKMAN US 51 STA 15+00 SPT"""	"""R080517A.cor"""	"""geotech2"""	-89.00332086	36.75546664	80.959
172	"""Borings"""	"""HICKMAN US 51 STA 15+00AC"""	"""R080517A.cor"""	"""geotech2"""	-89.00330081	36.75538135	78.054
173	"""Borings"""	"""HICKMAN US 51 STA 15+00 ST-2"""	"""R080517A.cor"""	"""geotech2"""	-89.00330451	36.75536655	79.168
174	"""Borings"""	"""HICKMAN US 51 STA 19+25 CBR"""	"""R080517A.cor"""	"""geotech2"""	-89.00348231	36.75668766	75.711
175	"""Borings"""	"""HICKMAN US 51 STA 19+25 SPT"""	"""R080517A.cor"""	"""geotech2"""	-89.00348055	36.75666883	79.104
176	"""Borings"""	"""HICKMAN US 51 STA 19+25 SP-2"""	"""R080517A.cor"""	"""geotech2"""	-89.00347084	36.75657325	79.722
177	"""Borings"""	"""HICKMAN US 51 STA 19+25 C ST-1"""	"""R080517A.cor"""	"""geotech2"""	-89.00346774	36.7565707	77.207
178	"""Borings"""	"""HICKMAN US 51 STA 30+00 ST-2"""	"""R080517A.cor"""	"""geotech2"""	-89.00389064	36.759598	76.591
179	"""Borings"""	"""HICKMAN US 51 STA 30+00 CBR"""	"""R080517A.cor"""	"""geotech2"""	-89.00388921	36.75957929	75.495
180	"""Borings"""	"""HICKMAN US 51 STA 30+00 SPT"""	"""R080517A.cor"""	"""geotech2"""	-89.00388401	36.75956239	75.467
181	"""Borings"""	"""HICKMAN US 51 STA 30+00 AC ST1"""	"""R080517A.cor"""	"""geotech2"""	-89.00387825	36.75950997	74.916
182	"""Borings"""	"""HICKMAN US 51 STA 40+00 CBR"""	"""R080517A.cor"""	"""geotech2"""	-89.00427642	36.76234734	77.662
183	"""Borings"""	"""HICKMAN US 51 STA 40+00 SPT"""	"""R080517A.cor"""	"""geotech2"""	-89.00427378	36.76234151	76.826
184	"""Borings"""	"""HICKMAN US 51 STA 40+00 ST-2"""	"""R080517A.cor"""	"""geotech2"""	-89.00425916	36.7622744	76.087
185	"""Borings"""	"""HICKMAN US 51 STA 40+00 ST1 AC"""	"""R080517A.cor"""	"""geotech2"""	-89.00426161	36.7622633	77.824
186	"""Borings"""	"""HICKMAN US 51 STA 50+00 ST-2"""	"""R080517A.cor"""	"""geotech2"""	-89.00465145	36.76511418	75.373
187	"""Borings"""	"""HICKMAN US 51 STA 50+00 CBR"""	"""R080517A.cor"""	"""geotech2"""	-89.00465731	36.7651106	74.383
188	"""Borings"""	"""HICKMAN US 51 STA 50+00 SPT"""	"""R080517A.cor"""	"""geotech2"""	-89.00465423	36.76509118	74.991
189	"""Borings"""	"""HICKMAN US 51 STA 50+00 AC ST1"""	"""R080517A.cor"""	"""geotech2"""	-89.00464368	36.765004	75.624
190	"""Borings"""	"""US 62 McCRACKEN 60+00 AC ST-1"""	"""R081117A.cor"""	"""geotech2"""	-88.68728857	37.04806861	93.47
191	"""Borings"""	"""US 62 McCRACKEN 60+00 SPT"""	"""R081117A.cor"""	"""geotech2"""	-88.68726997	37.0480692	93.941
192	"""Borings"""	"""US 62 McCRACKEN 60+00 CBR"""	"""R081117A.cor"""	"""geotech2"""	-88.68717803	37.04808888	94.144
193	"""Borings"""	"""US 62 McCRACKEN 50+00 CBR"""	"""R081117A.cor"""	"""geotech2"""	-88.69064812	37.04734297	94.881
194	"""Borings"""	"""US 62 McCRACKEN 50+00 AC ST-1"""	"""R081117A.cor"""	"""geotech2"""	-88.6905595	37.04735611	95.966
195	"""Borings"""	"""US 62 McCRACKEN 50+00 SPT"""	"""R081117A.cor"""	"""geotech2"""	-88.69066447	37.04733762	98.254
196	"""Borings"""	"""US 62 McCRACKEN 40+00 SPT"""	"""R081117A.cor"""	"""geotech2"""	-88.69380354	37.0467137	102.936
197	"""Borings"""	"""US 62 McCRACKEN 40+00 CBR"""	"""R081117A.cor"""	"""geotech2"""	-88.6937815	37.04670978	104.819
198	"""Borings"""	"""US 62 McCRACKEN 40+00 AC ST1"""	"""R081117A.cor"""	"""geotech2"""	-88.69391054	37.04669066	106.188
199	"""Borings"""	"""US 62 McCRACKEN 0+00 AC ST1"""	"""R081117A.cor"""	"""geotech2"""	-88.7070983	37.04388281	96.01
200	"""Borings"""	"""US 62 McCRACKEN 0+00 SPT"""	"""R081117A.cor"""	"""geotech2"""	-88.70700725	37.04393385	96.386

Table B-5. GPS positions of Tested Sites

ID	Attributes	(recorded in datalogger)	File name (rover)	Workspace	Longitude (DD)	Latitude (DD	Elevation (HAE)
201	"""Borings"""	"""US 62 McCRACKEN 0+00 CBR"""	"""R081117A.cor"""	"""geotech2"""	-88.70699327	37.04394658	94.676
202	"""Borings"""	"""TRIGG US 68 170+00 AC ST1"""	"""R082612A.cor"""	"""geotech2"""	-87.73765554	36.88024499	130.663
203	"""Borings"""	"""TRIGG US 68 170+00 ST2"""	"""R082612A.cor"""	"""geotech2"""	-87.73764433	36.88025429	130.735
204	"""Borings"""	"""TRIGG US 68 170+00SPT"""	"""R082612A.cor"""	"""geotech2"""	-87.73756382	36.88031581	131.269
205	"""Borings"""	"""TRIGG US 68 170+00 CBR"""	"""R082612A.cor"""	"""geotech2"""	-87.7375664	36.88031444	131.135
206	"""Borings"""	"""TRIGG US 68 140+00 CBR"""	"""R082612A.cor"""	"""geotech2"""	-87.74646602	36.87682865	134.12
207	"""Borings"""	"""TRIGG US 68 140+00 SPT"""	"""R082612A.cor"""	"""geotech2"""	-87.74644888	36.8768313	133.546
208	"""Borings"""	"""TRIGG US 68 140+00 ST1"""	"""R082612A.cor"""	"""geotech2"""	-87.74648542	36.87683001	133.381
209	"""Borings"""	"""TRIGG US 68 110+00 CBR"""	"""R082612A.cor"""	"""geotech2"""	-87.75628816	36.8746264	133.059
210	"""Borings"""	"""TRIGG US 68 110+00 SPT"""	"""R082612A.cor"""	"""geotech2"""	-87.75626864	36.87463139	132.782
211	"""Borings"""	"""TRIGG US 68 110+00 ST1"""	"""R082612A.cor"""	"""geotech2"""	-87.75619077	36.87466002	132.364
212	"""Borings"""	"""TRIGG US 68 110+00 ST2 AC CORE"""	"""R082612A.cor"""	"""geotech2"""	-87.75618714	36.87466703	132.917
213	"""Borings"""	"""TRIGG US 68 90+00 ST1"""	"""R082612A.cor"""	"""geotech2"""	-87.76244888	36.87232431	128.161
214	"""Borings"""	"""TRIGG US 68 90+00 CBR"""	"""R082612A.cor"""	"""geotech2"""	-87.76243587	36.87232658	127.726
215	"""Borings"""	"""TRIGG US 68 90+00 SPT"""	"""R082612A.cor"""	"""geotech2"""	-87.76242054	36.87233277	127.793
216	"""Borings"""	"""TRIGG US 68 90+00 CORE"""	"""R082612A.cor"""	"""geotech2"""	-87.76232833	36.87236573	127.756
217	"""Borings"""	"""TRIGG US 68 60+00 CBR"""	"""R082612A.cor"""	"""geotech2"""	-87.77205527	36.86955459	126.3
218	"""Borings"""	"""TRIGG US 68 60+00 SPT"""	"""R082612A.cor"""	"""geotech2"""	-87.77203385	36.8695615	126.447
219	"""Borings"""	"""TRIGG US 68 60+00 CORE ST1"""	"""R082612A.cor"""	"""geotech2"""	-87.77193388	36.86959063	126.422
220	"""Borings"""	"""TRIGG US 68 30+00 CBR"""	"""R082612A.cor"""	"""geotech2"""	-87.78228652	36.86780678	123.214
221	"""Borings"""	"""TRIGG US 68 30+00 SPT"""	"""R082612A.cor"""	"""geotech2"""	-87.78225788	36.8678014	124.223
222	"""Borings"""	"""TRIGG US 68 30+00 ST1"""	"""R082612A.cor"""	"""geotech2"""	-87.7821624	36.86779873	124.082
223	"""Borings"""	"""TRIGG US 68 30+00 ST2"""	"""R082612A.cor"""	"""geotech2"""	-87.78215776	36.86780354	123.572
224	"""Borings"""	"""DAVIESS KY 331 STA 2+00 AC ST1"""	"""R090116A.cor"""	"""geotech2"""	-87.1560728	37.78492162	98.289
225	"""Borings"""	"""DAVIESS KY 331 STA 2+00 SPT"""	"""R090116A.cor"""	"""geotech2"""	-87.15605501	37.78499602	98.442
226	"""Borings"""	"""DAVIESS KY 331 STA 2+00 CBR"""	"""R090116A.cor"""	"""geotech2"""	-87.15605296	37.7850096	97.675
227	"""Borings"""	"""DAVIESS KY 331 STA ST1 AC CORE"""	"""R090118A.cor"""	"""geotech2"""	-87.15567325	37.78618871	101.782
228	"""Borings"""	"""DAVIESS KY 331 STA 7+00 SPT"""	"""R090118A.cor"""	"""geotech2"""	-87.15564834	37.78625358	104.183
229	"""Borings"""	"""DAVIESS KY 331 STA 7+00 CBR"""	"""R090118A.cor"""	"""geotech2"""	-87.15564089	37.78626914	104.174
230	"""Borings"""	"""DAVIESS KY 331 STA 14+00 AC CR"""	"""R090118A.cor"""	"""geotech2"""	-87.1537184	37.78735754	105.066
231	"""Borings"""	"""DAVIESS KY 331 STA 14+00 SPT"""	"""R090118A.cor"""	"""geotech2"""	-87.15361569	37.78735892	106.523
232	"""Borings"""	"""DAVIESS KY 331 STA 14+00 CBR"""	"""R090118A.cor"""	"""geotech2"""	-87.15360149	37.78735435	104.906
233	"""Borings"""	"""DAVIESS KY 331 STA 20+00ST1 CR"""	"""R090118A.cor"""	"""geotech2"""	-87.15156574	37.78721959	108.057
234	"""Borings"""	"""DAVIESS KY 331 STA 20+00 SPT"""	"""R090118A.cor"""	"""geotech2"""	-87.15146776	37.78724532	108.001
235	"""Borings"""	"""DAVIESS KY 331 STA 20+00 CBR"""	"""R090118A.cor"""	"""geotech2"""	-87.15144623	37.78724981	107.39
236	"""Borings"""	"""DAVIESS KY 331 STA 32+00 CBR"""	"""R090118A.cor"""	"""geotech2"""	-87.14997257	37.78973902	113.282
237	"""Borings"""	"""DAVIESS KY 331 STA 32+00 ST1"""	"""R090118A.cor"""	"""geotech2"""	-87.14997262	37.78974646	113.678
238	"""Borings"""	"""DAVIESS KY 331 STA 32+00 SPT"""	"""R090118A.cor"""	"""geotech2"""	-87.14997278	37.78976951	114.005
239	"""Borings"""	"""DAVIESS KY 331 40+00 SPT"""	"""R090118A.cor"""	"""geotech2"""	-87.15020828	37.79187906	110.683
240	"""Borings"""	"""DAVIESS KY 331 40+00 CBR"""	"""R090118A.cor"""	"""geotech2"""	-87.15021341	37.79189469	111.292
241	"""Borings"""	"""DAVIESS KY 331 40+00 ST1"""	"""R090118A.cor"""	"""geotech2"""	-87.15021511	37.7919222	107.215
242	"""Borings"""	"""DAVIESS KY 331 2+00 AC ST1"""	"""R091317A.cor"""	"""geotech2"""	-87.1560594	37.78491636	100.867
243	"""Borings"""	"""DAVIESS KY 331 2+00 AC SPT"""	"""R091317A.cor"""	"""geotech2"""	-87.15603738	37.78498914	101.107
244	"""Borings"""	"""DAVIESS KY 331 2+00 AC CBR"""	"""R091317A.cor"""	"""geotech2"""	-87.15603572	37.78499997	102.05
245	"""Borings"""	"""BRECK US 60 10+00 AC ST CORE"""	"""R091613A.cor"""	"""geotech2"""	-86.47783172	37.77492137	167.421
246	"""Borings"""	"""BRECK US 60 10+00 SPT"""	"""R091613A.cor"""	"""geotech2"""	-86.47779719	37.77484097	167.417
247	"""Borings"""	"""BRECK US 60 10+00 CBR"""	"""R091613A.cor"""	"""geotech2"""	-86.47778731	37.7748255	167.55
248	"""Borings"""	"""BRECK US 60 40+00 ST1"""	"""R091613A.cor"""	"""geotech2"""	-86.47436666	37.76711063	174.506
249	"""Borings"""	"""BRECK US 60 40+00 SPT"""	"""R091613A.cor"""	"""geotech2"""	-86.47433753	37.76703445	174.06
250	"""Borings"""	"""BRECK US 60 40+00 CBR"""	"""R091613A.cor"""	"""geotech2"""	-86.47433457	37.76701711	174.046
251	"""Borings"""	"""BRECK US 60 70+00 CBR"""	"""R091613A.cor"""	"""geotech2"""	-86.46663646	37.76286469	167.77

Table B-6. GPS positions of Tested Sites

ID	Attributes	(recorded in datalogger)	File name (rover)	Workspace	Longitude (DD)	Latitude (DD	Elevation (HAE)
252	"""Borings"""	"""BRECK US 60 70+00 SPT"""	"""R091613A.cor"""	"""geotech2"""	-86.46667153	37.76286968	167.383
253	"""Borings"""	"""BRECK US 60 70+00 ST2"""	"""R091613A.cor"""	"""geotech2"""	-86.46675866	37.76287947	168.033
254	"""Borings"""	"""BRECK US 60 70+00 AC ST2"""	"""R091613A.cor"""	"""geotech2"""	-86.46676662	37.76287852	167.835
255	"""Borings"""	"""BRECK US 60 160+00 CORE ST1"""	"""R091613A.cor"""	"""geotech2"""	-86.43656845	37.7590667	185.223
256	"""Borings"""	"""BRECK US 60 160+00 SPT"""	"""R091613A.cor"""	"""geotech2"""	-86.43648981	37.75900892	185.042
257	"""Borings"""	"""BRECK US 60 160+00 CBR"""	"""R091613A.cor"""	"""geotech2"""	-86.43647154	37.758996	184.754
258	"""Borings"""	"""BRECK US 60 200+00 SPT"""	"""R091613A.cor"""	"""geotech2"""	-86.42444203	37.75361192	192.384
259	"""Borings"""	"""BRECK US 60 200+00 CBR"""	"""R091613A.cor"""	"""geotech2"""	-86.42441998	37.7536053	191.9
260	"""Borings"""	"""BRECK US 60 200+00 ST1"""	"""R091613A.cor"""	"""geotech2"""	-86.42440726	37.75360097	191.419
261	"""Borings"""	"""BRECK US 60 120+00 CBR"""	"""R091613A.cor"""	"""geotech2"""	-86.44971785	37.76140604	178.63
262	"""Borings"""	"""BRECK US 60 120+00 SPT"""	"""R091613A.cor"""	"""geotech2"""	-86.449736	37.76140456	178.531
263	"""Borings"""	"""BRECK US 60 120+00 CORE ST1"""	"""R091613A.cor"""	"""geotech2"""	-86.44981218	37.76140979	178.235
264	"""Borings"""	"""HARDIN US62 13.7 ST-1"""	"""R100718A.cor"""	"""geotech2"""	-85.93002149	37.66365715	188.812
265	"""Borings"""	"""HARDIN US62 13.7 ST-2"""	"""R100718A.cor"""	"""geotech2"""	-85.93001053	37.6636617	188.426
266	"""Borings"""	"""HARDIN US62 13.7 SPT"""	"""R100718A.cor"""	"""geotech2"""	-85.9299223	37.66370382	188.474
267	"""Borings"""	"""HARDIN US62 13.7 CBR"""	"""R100718A.cor"""	"""geotech2"""	-85.92990493	37.66371233	188.621
268	"""Borings"""	"""HARDIN US62 12.5 CORE ST1"""	"""R100718A.cor"""	"""geotech2"""	-85.94414453	37.65503313	186.812
269	"""Borings"""	"""HARDIN US62 12.5 SPT"""	"""R100718A.cor"""	"""geotech2"""	-85.94407648	37.65509437	187.322
270	"""Borings"""	"""HARDIN US62 12.5 CBR"""	"""R100718A.cor"""	"""geotech2"""	-85.94406705	37.65511233	187.575
271	"""Borings"""	"""HARDIN US62 12.90 ST1"""	"""R100718A.cor"""	"""geotech2"""	-85.93899499	37.65884068	179.962
272	"""Borings"""	"""HARDIN US62 12.90 ST2"""	"""R100718A.cor"""	"""geotech2"""	-85.93897931	37.65884624	180.597
273	"""Borings"""	"""HARDIN US62 12.90 Spt"""	"""R100718A.cor"""	"""geotech2"""	-85.93890709	37.65889409	181.444
274	"""Borings"""	"""HARDIN US62 12.90 CBR"""	"""R100718A.cor"""	"""geotech2"""	-85.93889177	37.6589076	181.804
275	"""Borings"""	"""HARDIN US62 12.80 CBR"""	"""R100718A.cor"""	"""geotech2"""	-85.94027393	37.65833971	182.846
276	"""Borings"""	"""HARDIN US62 12.80 core ST1"""	"""R100718A.cor"""	"""geotech2"""	-85.94029739	37.65831869	183.508
277	"""Borings"""	"""HARDIN US62 12.80 SPT"""	"""R100718A.cor"""	"""geotech2"""	-85.94037364	37.65827195	183.641
278	"""Borings"""	"""HARDIN US62 12.45 CORE ST1"""	"""R100718A.cor"""	"""geotech2"""	-85.94484465	37.65469381	189.711
279	"""Borings"""	"""HARDIN US62 12.4 SPT"""	"""R100718A.cor"""	"""geotech2"""	-85.94490323	37.6546309	190.888
280	"""Borings"""	"""HARDIN US62 12.4 CBR"""	"""R100718A.cor"""	"""geotech2"""	-85.944918	37.65461303	190.894
281	"""Borings"""	"""HARDIN US62 12.2 CORE ST-1"""	"""R100718A.cor"""	"""geotech2"""	-85.94747428	37.6520462	183.194
282	"""Borings"""	"""HARDIN US62 12.2 SPT"""	"""R100718A.cor"""	"""geotech2"""	-85.94753566	37.6519855	183.477
283	"""Borings"""	"""HARDIN US62 12.2 CBR"""	"""R100718A.cor"""	"""geotech2"""	-85.94754946	37.65197224	184.317
284	"""Borings"""	"""HARDIN US62 12.0 CORE ST1"""	"""R100718A.cor"""	"""geotech2"""	-85.94953225	37.64959773	183.189
285	"""Borings"""	"""HARDIN US62 12.0 SPT"""	"""R100718A.cor"""	"""geotech2"""	-85.94946688	37.64966166	183.535
286	"""Borings"""	"""HARDIN US62 12.0 CBR"""	"""R100718A.cor"""	"""geotech2"""	-85.94946011	37.64966292	180.918
287	"""Borings"""	"""HARDIN US 62 0+00 13.75 CR ST1"""	"""R100719A.cor"""	"""geotech2"""	-85.92564158	37.66584313	196.734
288	"""Borings"""	"""HARDIN US 62 0+00 13.75 ST2"""	"""R100719A.cor"""	"""geotech2"""	-85.92562852	37.66584825	199.99
289	"""Borings"""	"""HARDIN US 62 0+00 13.75 SPT"""	"""R100719A.cor"""	"""geotech2"""	-85.92555742	37.6658815	199.881
290	"""Borings"""	"""HARDIN US 62 0+00 13.75 CBR"""	"""R100719A.cor"""	"""geotech2"""	-85.92553389	37.6658937	200.153
291	"""Borings"""	"""HARDIN US62 10+00 13.95EB ST1"""	"""R100719A.cor"""	"""geotech2"""	-85.92269018	37.66718262	192.898
292	"""Borings"""	"""HARDIN US62 10+00 13.95EB ST2"""	"""R100719A.cor"""	"""geotech2"""	-85.92267599	37.66719392	191.986
293	"""Borings"""	"""HARDIN US6210+00 13.95EB SPT"""	"""R100719A.cor"""	"""geotech2"""	-85.92259417	37.66722187	192.293
294	"""Borings"""	"""HARDIN US62 10+00 13.75EB CBR"""	"""R100719A.cor"""	"""geotech2"""	-85.92257985	37.66722934	192.321
295	"""Borings"""	"""HARDIN US62 41+50 14.6WB ACST1"""	"""R100719A.cor"""	"""geotech2"""	-85.91269863	37.67142059	183.232
296	"""Borings"""	"""HARDIN US62 41+50 14.6WB SPT"""	"""R100719A.cor"""	"""geotech2"""	-85.91278911	37.67139029	182.359
297	"""Borings"""	"""HARDIN US62 41+50 14.6WB CBR"""	"""R100719A.cor"""	"""geotech2"""	-85.91280223	37.67138384	182.356
298	"""Borings"""	"""HARDIN US62 38+50 14.5WB ACST1"""	"""R100719A.cor"""	"""geotech2"""	-85.9137624	37.67098472	181.093
299	"""Borings"""	"""HARDIN US62 38+50 14.5WB SPT"""	"""R100719A.cor"""	"""geotech2"""	-85.91386003	37.67094668	180.765
300	"""Borings"""	"""HARDIN US62 38+50 14.5WB CBR"""	"""R100719A.cor"""	"""geotech2"""	-85.91390812	37.67094694	180.697
301	"""Borings"""	"""HARDIN US62 20+00 14.2WB ACST1"""	"""R100719A.cor"""	"""geotech2"""	-85.91947135	37.66872418	185.37
302	"""Borings"""	"""HARDIN US62 20+00 14.2WB SPT"""	"""R100719A.cor"""	"""geotech2"""	-85.91956409	37.66868807	185.632
303	"""Borings"""	"""HARDIN US62 20+00 14.2WB CBR"""	"""R100719A.cor"""	"""geotech2"""	-85.91957898	37.66868482	185.631

Table B-7. GPS positions of Tested Sites

ID	Attributes	(recorded in datalogger)	File name (rover)	Workspace	Longitude (DD)	Latitude (DD	Elevation (HAE)
304	"""Borings"""	"""HARDIN US62 10+00 13.95WBACST1"""	"""R100719A.cor"""	"""geotech2"""	-85.92259399	37.66746452	192.935
305	"""Borings"""	"""HARDIN US62 10+00 13.95WB SPT"""	"""R100719A.cor"""	"""geotech2"""	-85.92269	37.66743074	192.731
306	"""Borings"""	"""HARDIN US62 10+00 13.95WB CBR"""	"""R100719A.cor"""	"""geotech2"""	-85.92270386	37.66742501	192.491
307	"""Borings"""	"""HARDIN US62 0+00 13.75WB ACST1"""	"""R100719A.cor"""	"""geotech2"""	-85.92606637	37.66590157	198.902
308	"""Borings"""	"""HARDIN US62 0+00 13.75WB SPT"""	"""R100719A.cor"""	"""geotech2"""	-85.92615713	37.66586662	198.096
309	"""Borings"""	"""HARDIN US62 0+00 13.75WB CBR"""	"""R100719A.cor"""	"""geotech2"""	-85.9261763	37.66585049	198.882
310	"""Borings"""	OWEN 127 0+00 14.3 NB ST2	"""R101415A.cor"""	"""geotech2"""	-84.84099186	38.51400478	238.465
311	"""Borings"""	"""OWEN 127 0+00 14.3 NB CPT"""	"""R101415A.cor"""	"""geotech2"""	-84.840971	38.51393174	238.981
312	"""Borings"""	"""OWEN 127 0+00 14.3 NB ST2"""	"""R101415A.cor"""	"""geotech2"""	-84.84091928	38.51403231	238.304
313	"""Borings"""	"""OWEN 127 0+00 14.3 NB AC ST1"""	"""R101415A.cor"""	"""geotech2"""	-84.8409219	38.5140428	240.246
314	"""Borings"""	"""OWEN 127 0+00 14.3 NB SPT"""	"""R101415A.cor"""	"""geotech2"""	-84.84088822	38.51412536	240.673
315	"""Borings"""	"""OWEN 127 0+00 14.3 NB CBR"""	"""R101415A.cor"""	"""geotech2"""	-84.84087578	38.51414536	240.307
316	"""Borings"""	"""OWEN 12710+00 14.5 SB CORE"""	"""R101415A.cor"""	"""geotech2"""	-84.84016032	38.51652206	237.153
317	"""Borings"""	"""OWEN 12710+00 14.5 SB AC CORE"""	"""R101415A.cor"""	"""geotech2"""	-84.84016104	38.51654189	239.529
318	"""Borings"""	"""OWEN 12710+00 14.5 SB SPT"""	"""R101415A.cor"""	"""geotech2"""	-84.84014954	38.51658457	238.864
319	"""Borings"""	"""OWEN 12710+00 14.5 SB CBR"""	"""R101415A.cor"""	"""geotech2"""	-84.84014286	38.51660815	239.523
320	"""Borings"""	"""OWEN 127 20+00 14.7 NB CPT"""	"""R101415A.cor"""	"""geotech2"""	-84.83844677	38.51880943	239.952
321	"""Borings"""	"""OWEN 127 20+00 14.7 NB ST2"""	"""R101415A.cor"""	"""geotech2"""	-84.8384302	38.51882237	240.635
322	"""Borings"""	"""OWEN 127 20+00 14.7 NB ACST1"""	"""R101415A.cor"""	"""geotech2"""	-84.8384229	38.51883204	240.573
323	"""Borings"""	"""OWEN 127 20+00 14.7 NB SPT"""	"""R101415A.cor"""	"""geotech2"""	-84.83834792	38.51890278	240.917
324	"""Borings"""	"""OWEN 127 20+00 14.7 NB SPT"""	"""R101415A.cor"""	"""geotech2"""	-84.83834921	38.5189055	241.791
325	"""Borings"""	"""OWEN 127 20+00 14.7 NB CBR"""	"""R101415A.cor"""	"""geotech2"""	-84.83833368	38.51892104	241.551
326	"""Borings"""	"""OWEN 127 30+00 14.9 SB ST2"""	"""R101415A.cor"""	"""geotech2"""	-84.83633848	38.52092053	246.385
327	"""Borings"""	"""OWEN 127 30+00 14.9 SB CBR"""	"""R101415A.cor"""	"""geotech2"""	-84.83632179	38.52093936	245.949
328	"""Borings"""	"""OWEN 127 30+00 14.9 SB SPT"""	"""R101415A.cor"""	"""geotech2"""	-84.83631974	38.52095207	246.969
329	"""Borings"""	"""OWEN 127 30+00 14.9 SB AC ST1"""	"""R101415A.cor"""	"""geotech2"""	-84.83630686	38.52095626	247.549
330	"""Borings"""	"""OWEN 127 40+00 15.1 NB ST2"""	"""R101415A.cor"""	"""geotech2"""	-84.8355876	38.5237258	243.897
331	"""Borings"""	"""OWEN 127 40+00 15.1 NB AC ST1"""	"""R101415A.cor"""	"""geotech2"""	-84.83559095	38.5237397	245.389
332	"""Borings"""	"""OWEN 127 40+00 15.1 NB AC SPT"""	"""R101415A.cor"""	"""geotech2"""	-84.8355833	38.52382228	246.37
333	"""Borings"""	"""OWEN 127 40+00 15.1 NB CBR"""	"""R101415A.cor"""	"""geotech2"""	-84.83557963	38.52384142	249.738
334	"""Borings"""	"""OWEN 127 50+00 15.3 SB ST1"""	"""R101415A.cor"""	"""geotech2"""	-84.83418377	38.52622801	260.393
335	"""Borings"""	"""OWEN 127 50+00 15.3 SB ST2"""	"""R101415A.cor"""	"""geotech2"""	-84.83419096	38.52621527	258.839
336	"""Borings"""	"""OWEN 127 50+00 15.3 SB SPT"""	"""R101415A.cor"""	"""geotech2"""	-84.83426412	38.52614613	258.414
337	"""Borings"""	"""OWEN 127 50+00 15.3 SB CBR"""	"""R101415A.cor"""	"""geotech2"""	-84.83427427	38.52613821	258.045
338	"""Borings"""	"""DAVIESS KY 331 STA 2+00 AC ST1"""	"""ky331.cor"""	"""geotech2"""	-87.1560728	37.78492162	98.289
339	"""Borings"""	"""DAVIESS KY 331 STA 2+00 SPT"""	"""ky331.cor"""	"""geotech2"""	-87.15605501	37.78499602	98.442
340	"""Borings"""	"""DAVIESS KY 331 STA 2+00 CBR"""	"""ky331.cor"""	"""geotech2"""	-87.15605296	37.7850096	97.675
341	"""Borings"""	"""DAVIESS KY 331 STA ST1 AC CORE"""	"""ky331.cor"""	"""geotech2"""	-87.15567325	37.78618871	101.782
342	"""Borings"""	"""DAVIESS KY 331 STA 7+00 SPT"""	"""ky331.cor"""	"""geotech2"""	-87.15564834	37.78625358	104.183
343	"""Borings"""	"""DAVIESS KY 331 STA 7+00 CBR"""	"""ky331.cor"""	"""geotech2"""	-87.15564089	37.78626914	104.174
344	"""Borings"""	"""DAVIESS KY 331 STA 14+00 AC CR"""	"""ky331.cor"""	"""geotech2"""	-87.1537184	37.78735754	105.066
345	"""Borings"""	"""DAVIESS KY 331 STA 14+00 SPT"""	"""ky331.cor"""	"""geotech2"""	-87.15361569	37.78735892	106.523
346	"""Borings"""	"""DAVIESS KY 331 STA 14+00 CBR"""	"""ky331.cor"""	"""geotech2"""	-87.15360149	37.78735435	104.906
347	"""Borings"""	"""DAVIESS KY 331 STA 20+00ST1 CR"""	"""ky331.cor"""	"""geotech2"""	-87.15156574	37.78721959	108.057
348	"""Borings"""	"""DAVIESS KY 331 STA 20+00 SPT"""	"""ky331.cor"""	"""geotech2"""	-87.15146776	37.78724532	108.001
349	"""Borings"""	"""DAVIESS KY 331 STA 20+00 CBR"""	"""ky331.cor"""	"""geotech2"""	-87.15144623	37.78724981	107.39
350	"""Borings"""	"""DAVIESS KY 331 STA 32+00 CBR"""	"""ky331.cor"""	"""geotech2"""	-87.14997257	37.78973902	113.282
351	"""Borings"""	"""DAVIESS KY 331 STA 32+00 ST1"""	"""ky331.cor"""	"""geotech2"""	-87.14997262	37.78974646	113.678
352	"""Borings"""	"""DAVIESS KY 331 STA 32+00 SPT"""	"""ky331.cor"""	"""geotech2"""	-87.14997278	37.78976951	114.005
353	"""Borings"""	"""DAVIESS KY 331 40+00 SPT"""	"""ky331.cor"""	"""geotech2"""	-87.15020828	37.79187906	110.683
354	"""Borings"""	"""DAVIESS KY 331 40+00 CBR"""	"""ky331.cor"""	"""geotech2"""	-87.15021341	37.79189469	111.292
355	"""Borings"""	"""DAVIESS KY 331 40+00 ST1"""	"""ky331.cor"""	"""geotech2"""	-87.15021511	37.7919222	107.215

Appendix C

Index properties of untreated soils and soils mixed with chemical admixtures

Table C-1. Index properties of stabilized and non-stabilized subgrades.

County/Rt.	*Station or Mile*	*Method of Stab.*	*L.L. Stab.*	*P.L. Stab.*	*P.I. Stab*	*L.L. Non-Stab.*	*P.L. Non-Stab.*	*P.I. Non-Stab,.*	*S.G. Stab.*	*S.G. Non-Stab.*	*Class Stab. ASSHTO*	*Class Non-Stab ASSHTO*	*Class Stab. UCS*	*Class Non-Stab UCS*
Anderson US 127	11+50	Lime	38.8	30.9	7.9				2.62		A-4		ML	
Anderson US 127	30+00	Lime	36.5	25.2	11.3	31.8	19.1	12.7	2.75	2.79	A-6	A-6	ML	CL
Anderson US 127	50+00	Lime	30.8	23.1	7.7	41.9	20.4	21.5	2.7	2.74	A-4	A-7-6	ML	SC
Anderson US 127	65+64	Lime	NP	NP	NP	NP	NP	NP	2.73	2.65	A-4	A-4	SM	SM
Anderson US 127	75+38	Lime	27.0	20.3	6.7	18.9	13.1	5.8	2.67	2.60	A-4	A-4	CL-ML	SM
Boone KY 842	10+00	Lime/Cem.	NP	NP	NP	30.5	17.0	13.5	2.54	2.62	A-4	A-6	ML	CL
Boone KY 842	30+00	Lime/Cem.	36.0	29.0	7.0	48.0	18.4	29.6	2.65	2.59	A-4	A-7-6	ML	CL
Boone KY 842	50+00	Lime/Cem.	NP	NP	NP				2.61		A-4		SM	
Boone KY 842	70+00	Lime/Cem.	41.0	34.2	6.8				2.65		A-5		ML	
Boone KY 842	90+00	Lime/Cem.	40.0	28.6	11.4				2.62		A-6		ML	
Boone KY 842	110+00	Lime/Cem.				39.0	21.5	17.5		2.63		A-6		CL
Boone KY 842	120+00	Lime/Cem.	36.2	25.8	10.4				2.62		A-6		ML	
Boyle US 127	14+00	Lime	33.1	29.3	3.8				2.89		A-4		ML	
Boyle US 127	25+00	Lime	40.9	33.8	7.1	59.7	27.1	32.6	2.91	2.92	A-5	A-7-6	ML	CH
Boyle US 127	50+00	Lime	41.9	27.3	14.6				2.90		A-7-6		ML	
Boyle US 127	60+00	Lime	50.0	36.9	13.1	72.5	31.6	40.9	2.94	2.97	A-7-5	A-7-5	MH	CH
Boyle US 127	75+00	Lime				48.3	24.2	24.1		2.86		A-7-6		CL
Boyle US 127	100+00	Lime				30.2	23.3	6.9		2.79		A-4		ML
Breck US 60	10+00	Cement	27.9	27.1	0.8				2.86		A-4		ML	
Breck US 60	14+00	Cement												
Breck US 60	40+00	Cement				28.7	18.4	10.3		2.88		A-6		CL
Breck US 60	60+00	Cement												
Breck US 60	70+00	Grey Clay				39.2	21.7	17.5		2.90		A-6		SC
Breck US 60	70+00	B.rn Clay				35.3	22.1	13.2		2.87		A-6		CL
Breck US 60	120+00	Cement	28.8	28.2	0.6				2.85		A-4		ML	
Breck US 60	160+00	Cement												
Breck US 60	200+00	Cement				25.4	16.9	8.5		2.84		A-4		CL

Table C-2. Index properties of stabilized and non-stabilized subgrades.

County/ Rt.	*Station or mile*	*Method of Stab.*	*L.L. Stab.*	*P.L. Stab .*	*P.I. Stab*	*L.L. Non-Stab.*	*P.L. Non-Stab.*	*P.I. Non-Stab*	*S.G. Stab.*	*S.G. Non-Stab.*	*Class Stab. AASHTO*	*Class Non-Stab ASSHTO*	*Class Stab. UCS*	*Class Non-Stab UCS*
Hickman US 51	02+25	MKD	NP	NP	NP	NP	NP	NP	2.66	2.87	A-4	A-4	ML	ML
Hickman US 51	15+00	MKD	NP	NP	NP				2.70		A-4		ML	
Hickman US 51	19+25	MKD				NP	NP	NP		2.68		A-4		ML
Hickman US 51	22+50	MKD												
Hickman US 51	30+00	MKD				NP	NP	NP		2.65		A-4		ML
Hickman US 51	40+00	MKD	NP	NP	NP				2.88		A-4		ML	
Hickman US 51	50+00	MKD	NP	NP	NP	NP	NP	NP	2.66	2.68	A-4	A-4	ML	ML
Lee KY 11	10.0	NA												
Lee KY 11	10.2	NT				43.4	24.1	19.3		2.66		A-7-6		CL
Lee KY 11	10.4	NA												
Lee KY 11	10.5	10% Cement	NP	NP	NP				2.80		A-4		ML	
Lee KY 11	11.0	Lime	NP	NP	NP				2.63		A-4		SM	
Lee KY 11	11.2	Lime	NP	NP	NP				2.53		A-4		SM	
Lee KY 11	12.0	MKD	NP	NP	NP				2.62		A-4		SM	
Lee KY 11	12.5	MKD	NP	NP	NP				2.62		A-4		ML	
Lee KY 11	13.7	7% Cem.	NP	NP	NP				2.58		A-4		SM	
Lee KY 11	14.1	N/A				32.1	20.1	12		2.65		A-6		CL
Lee KY 11	14.5	A.F.B.C.	NP	NP	NP				2.77		A-4		ML	
Lee KY 11	14.7	A.F.B.C.	42.8	34.6	8.2				2.79		A-5		SM	
McCracken US 62	00+00	MKD				23.9	15.4	8.5		2.62		A-2-4		SC
McCracken US 62	10+00	MKD												
McCracken US 62	20+00	MKD	NP	NP	NP	25.7	15.0	10.7	2.71	2.67	A-4	A-2-4	ML	SC
McCracken US 62	30+00	MKD	NP	NP	NP	NP	NP	NP	2.72	2.69	A-4	ND2	ML	SM
McCracken US 62	40+00	MKD				24.2	14.5	9.7		2.87		A-2-4		SC
McCracken US 62	50+00	MKD	NP	NP	NP				2.62		A-4		SM	
McCracken US 62	60+00	MKD	NP	NP	NP	31.3	18.1	13.2	2.63	2.73	A-4	A-2-6	SM	SC

Table C-3. Index properties of stabilized and nonstabilized subgrades.

County/ Rt.	*Station or mile*	*Method of Stab.*	*L.L. Stab.*	*P.L. Stab.*	*P.I. Stab*	*L.L. Non-Stab.*	*P.L. Non-Stab.*	*P.I. Non-Stab*	*S.G. Stab.*	*S.G. Non-Stab.*	*Class Stab. AASHTO*	*Class Non-Stab ASSHTO*	*Class Stab. UCS*	*Class Non-Stab UCS*
Daviess KY 331	02+00	Cement				28.8	23.2	5.6		2.90		A-4		ML
Daviess KY 331	07+00	Cement	NP	NP	NP				2.89		A-4		ML	
Daviess KY 331	14+00	Cement	NP	NP	NP				2.89		A-4		SM	
Daviess KY 331	20+00	Cement	NP	NP	NP				2.90		A-4		ML	
Daviess KY 331	32+00	Cement				35.1	21.5	13.6		2.88		A-6		CL
Daviess KY 331	40+00	Cement				24.3	18.0	6.3		2.87		A-4		CL-ML
Fayette US 25	-04+00	Lime												
Fayette US 25	10+00	Lime				30.6	19.5	11.1		2.76		A-6		CL
Fayette US 25	20+00	Lime				84.6	39.9	44.7		2.90		A-7-5		MH
Fayette US 25	30+00	Lime												
Fayette US 25	40+00	Lime	39.1	28.3	10.8				2.74		A-6		ML	
Fayette US 25	50+00	Lime				42.5	23.3	19.2		2.71		A-7-6		CL
Fayette US 25	60+00	Lime				69.0	32.7	36.3		2.80		A-7-5		CH
Fayette US 25	70+00	Lime												
Fayette US 25	77+15	Lime	46.0	36.8	9.2				2.81		A-5		ML	
Fayette US 25	81+75	Lime	NP	NP	NP	42.8	26.8	16	2.67	2.87	A-4	A-7-6	ML	ML
Hardin US 62	12.0	Lime	NP	NP	NP	27.0	16.1	10.9	2.58	2.57	A-4	A-6	SM	CL
Hardin US 62	12.2	Lime												
Hardin US 62	12.45	Lime												
Hardin US 62	12.50	Lime	NP	NP	NP				2.70		A-2-4		SM	
Hardin US 62	12.8	Lime												
Hardin US 62	12.9	Lime												
Hardin US 62	13.7	Lime	NP	NP	NP				2.57		A-4		ML	
Hardin US 62	13.75E	Lime												
Hardin US 62	13.75W	Lime	NP	NP	NP	49.8	17.2	32.6	2.71	2.62	A-4	A-7-6	SM	CL
Hardin US 62	13.95E	Lime	NP	NP	NP				2.58		A-4		SM	
Hardin US 62	13.95W	Lime												
Hardin US 62	14.2	Lime												
Hardin US 62	14.5	Lime				35.0	14.0	21		2.58		A-6		CL
Hardin US 62	14.6	Lime				47.5	19.6	27.9		2.74		A-7-6		CL

Table C-4. Index properties of stabilized and nonstabilized subgrades.

County/ Rt.	*Station or milepost*	*Method of Stab.*	*L.L. Stab.*	*P.L. Stab.*	*P.I. Stab*	*L.L. Non-Stab.*	*P.L. Non-Stab.*	*P.I. Non-Stab*	*S.G. Stab.*	*S.G. Non-Stab.*	*Class Stab. AASHTO*	*Class Non-stab ASSHTO*	*Class Stab. UCS*	*Class Non-Stab UCS*
McCreary US 27	655+75	Cement	NP	NP	NP	NP	NP	NP	2.75	2.66	A-4	A-4	SM	SM
McCreary US 27	679+00	Cement	NP	NP	NP	NP	NP	NP	2.75	2.78	A-4	A-2-4	SM	SM
McCreary US 27	733+40	Cement	NP	NP	NP				2.76		A-1-b		SM	
McCreary US 27	756+30	Cement	NP	NP	NP	NP	NP	NP	2.75	2.61	A-4	A-4	ML	ML
McCreary US 27	774+20	Cement	NP	NP	NP				2.76		A-4		SM	
McCreary US 27	789+00	Cement	NP	NP	NP	NP	NP	NP	2.75	2.68	A-2-4	A-4	SM	SM
Owen US 127	00+00	Lime	43.5	31.3	12.2	41.3	22.9	18.4	2.77	2.73	A-7-5	A-7-6	ML	CL
Owen US 127	10+00	Lime												
Owen US 127	20+00	Lime				36.8	22.8	14	2.78	2.94	A-4	A-6	ML	CL
Owen US 127	30+00	Lime												
Owen US 127	40+00	Lime	38.0	31.1	6.9	43.7	23.2	20.5	2.78	2.77	A-4	A-7-6	ML	CL
Owen US 127	50+00	Lime												
Shelby KY 55	10+00	Lime	NP	NP	NP	36.9	19.6	17.3	2.76	2.76	A-4	A-6	ML	CL
Shelby KY 55	20+00	Lime	41.8	26.9	14.9	43.4	22.4	21	2.74	2.82	A-7-6	A-7-6	ML	CL
Shelby KY 55	30+75	Lime												
Shelby KY 55	40+00	Lime				46.8	21.2	25.6		2.74		A-7-6		CL
Shelby KY 55	50+00	Lime												
Shelby KY 55	60+00	Lime				31.9	21.4	10.5		2.71		A-6		CL
Trigg US 68	30+00	Lime	37.8	26.7	11.1	43.2	20.3	22.9	2.67	2.69	A-6	A-7-6	ML	CL
Trigg US 68	60+00	Lime	NP	NP	NP				2.65		A-4		SM	
Trigg US 68	90+00	Lime	NP	NP	NP	36.3	22.1	14.2	2.66	2.70	A-4	A-6	ML	CL
Trigg US 68	110+00	Lime	37.5	26.8	10.7	37.2	18.0	19.2	2.64	2.66	A-6	A-6	ML	CL
Trigg US 68	140+00	Lime	NP	NP	NP				2.67		A-4		ML	

Appendix D

Percent finer than No. 10 US sieve, No 200 US sieve, and 0.002-mm size particles for chemically treated subgrades and untreated subgrades

Table D-1. Results of grain –size analysis.

County/Treatment	*Station /Mile*	*% Passing #10 Sieve*	*%Passing #200 Sieve*	*%Passing .002mm Hydrometer*
Anderson US 127 Lime	11+50	98.61	72.91	19.89
Anderson US 127 Lime	30+00	99.60	79.05	18.72
Anderson US 127 NT[1]	30+00	100.00	81.70	25.83
Anderson US 127 Lime	50+00	97.02	56.56	15.84
Anderson US 127 NT	50+00	99.90	48.91	35.55
Anderson US 127 Lime	65+64	94.59	47.30	13.00
Anderson US 127 NT	65+64	98.46	37.01	16.83
Anderson US 127 Lime	75+30	98.27	51.51	14.09
Anderson US 127 NT	75+30	81.16	40.72	14.02
Boone KY 842 Lime/Cement	10+00wb	99.07	65.55	12.90
Boone KY 842 NT	10+00wb	100.00	87.03	29.84
Boone KY 842 Lime/Cement	30+00wb	98.09	68.84	16.26
Boone KY 842 NT	30+00wb	98.25	87.34	43.45
Boone KY 842 Lime/Cement	50+00wb	95.41	48.44	10.03
Boone KY 842 Lime/Cement	70+00wb	97.18	52.00	11.46
Boone KY 842 Lime/Cement	90+00wb	93.42	62.81	15.78
Boone KY 842 NT	110+00wb	66.68	61.78	30.34
Boone KY 842 Lime/Cement	120+00wb	99.26	72.37	16.41
Boyle US 127 Lime	14+00	96.55	62.66	17.20
Boyle US 127 Lime	25+00	97.87	66.76	24.51
Boyle US 127 NT	25+00	99.04	87.81	51.26
Boyle US 127 Lime	50+00	93.63	78.46	31.68
Boyle US 127 Lime	60+00	97.86	76.42	29.49
Boyle US 127 NT	60+00	98.48	90.51	55.65
Boyle US 127 NT	75+00	98.75	86.66	41.58
Boyle US 127 NT	100+00	83.91	70.07	21.11

1. NT—No treatment with chemical admixture

Table D-2. Results of grain –size analysis.

County	*Station /Mile*	*% Passing #10 Sieve*	*%Passing #200 Sieve*	*%Passing .002mm Hydrometer*
Hickman US 51 MKD	2+25	97.27	73.89	11.29
Hickman US 51 NT	2+25	99.44	80.67	13.18
Hickman US 52 MKD	15+00	97.08	68.69	9.12
Hickman US 51 NT	19+25	94.21	62.22	7.73
Hickman US 51 NT	30+00	97.12	86.12	12.82
Hickman US 51 MKD	40+00	92.94	56.32	8.25
Hickman US 51 MKD	50+00	99.01	66.01	7.39
Hickman US 51 NT	50+00	99.70	90.31	15.43
Lee KY 11 NT	10.2	97.04	87.66	36.70
Lee KY 11 Cement	10.5	88.33	50.46	11.33
Lee KY 11 Lime	11.0	83.03	40.21	6.02
Lee KY 11 Lime	11.2	86.59	46.85	13.27
Lee KY 11 MKD	12.0	85.70	42.86	8.83
Lee KY 11 MKD	12.5	95.69	58.96	15.40
Lee KY 11 7% Cement	13.7	87.90	44.14	8.84
Lee KY 11 NT	14.0	83.00	68.76	22.46
Lee KY 11 AFBC	14.50	89.97	50.27	7.22
Lee KY 11 AFBC	14.70	88.62	48.13	8.34
McCracken US 62 NT`	0+00	52.47	14.94	8.03
McCracken US 62 MKD	20+00	98.62	64.66	7.35
McCracken US 62 NT	20+00	59.88	22.82	10.92
McCracken US 62 MKD	30+00	87.32	68.34	11.63
McCracken US 62 NT	30+00	57.16	12.21	8.62
McCracken US 62 NT	40+00	86.03	23.93	14.57
McCracken US 62 MKD	50+00	79.09	45.92	7.77
McCracken US 62 MKD	60+00	85.91	44.63	6.51
McCracken US 62 NT	60+00	44.46	21.73	10.29

Table D-3. Results of grain –size analysis.

County	*Station /Mile*	*% Passing #10 Sieve*	*%Passing #200 Sieve*	*%Passing .002mm Hydrometer*
McCreary US 27 Cement	655+75	76.39	44.02	7.23
McCreary US 27 NT	655+75	85.52	45.80	10.71
McCreary US 27 Cement	679+00	90.01	40.38	8.35
McCreary US 27 NT	679+00	52.41	26.82	10.49
McCreary US 27 Cement	733+40	50.94	24.41	8.23
McCreary US 27 Cement	756+30	82.26	55.81	15.04
McCreary US 27 NT	756+30	94.25	77.50	32.06
McCreary US 27 Cement	774+20	87.31	41.26	9.41
McCreary US 27 Cement	789+00	73.75	31.03	4.44
McCreary US 27 NT	789+00	65.68	37.80	13.51
Owen US 127 Lime	0+00	98.23	62.34	16.57
Owen US 127 NT	0+00	99.53	98.09	52.83
Owen US 127 Lime	20+00	67.30	50.86	16.27
Owen US 127 NT	20+00	96.06	89.41	36.19
Owen US 127 Lime	40+00	95.49	73.75	20.45
Owen US 127 NT	40+00	96.82	88.76	38.47
Shelby KY 55 Lime	10+00	99.91	65.10	17.87
Shelby KY 55 NT	10+00	99.52	95.02	36.75
Shelby KY 55 Lime	20+00	99.91	82.71	27.94
Shelby KY 55 NT	20+00	99.98	85.44	38.88
Shelby KY 55 NT	40+00	99.61	93.02	44.55
Shelby KY 55 NT	60+00	100.00	94.19	25.40
Trigg US 68 Lime	30+00	98.95	85.14	18.00
Trigg US 68 NT	30+00	100.00	97.21	31.56
Trigg US 68 Lime	60+00	85.94	46.14	10.06
Trigg US 68 Lime	90+00	96.49	65.42	10.71
Trigg US 68 NT	90+00	100.00	97.45	27.69
Trigg US 68 Lime	110+00	91.12	72.04	15.85
Trigg US 68 NT	110+00	88.57	82.43	9.83
Trigg US 68 Lime	140+00	80.17	61.99	12.28
Trigg US 68 NT	170+00	99.75	86.18	36.28

APPENDIX E

Moisture Contents of Treated and Untreated Subgrades

Table E-1. Moisture contents of in situ CBR locations and resilient modulus specimens.

Anderson US 127		
Station	Moisture Content of In situ CBR Location (Percent)	Moisture Content of Resilient Modulus Specimen (Percent)
11+50 Treated	27.96	
11+50 Untreated	21.46	
30+00 Treated	24.64	21.15
30+00 Untreated	18.63	18.07
50+00 Treated	16.48	
50+00 Untreated	21.31	19.7
65+64 Treated	20.49	
75+38 Treated	20.18	18.45
75+38 Untreated	36.99	
75+38 Rock	4.08	

Table E-2. Moisture contents of in situ CBR locations and resilient modulus specimens.

Trigg US 68		
Station	Moisture Content of In situ CBR Location (Percent)	Moisture Content of Resilient Modulus Specimen (Percent)
30+00 Treated	21.32	
30+00 Untreated	19.96	19.55
60+00 Treated	26.02	29.27
60+00 Untreated	25.81	
90+00 Treated	20.77	24.33
90+00 Untreated	19.09	19.63
110+00 Treated	18.37	
110+00 Untreated	18.93	
140+00 treated	22.14	
140+00 Untreated	22.15	
170+00 Treated	24.70	
170+00 Untreated	22.90	

Table E-3. Moisture contents of in situ CBR locations and resilient modulus specimens.

Shelby KY 55		
Station	Moisture Content of In situ CBR Location (Percent)	Moisture Content of Resilient Modulus Specimen (Percent)
10+00 Treated	30.14	
10+00 Untreated	25.79	
20+00 treated	20.75	
20+00 Untreated	20.33	
30+75 Treated	26.90	
30+75 Untreated	26.31	
40+00 Treated	28.11	
40+00 Untreated	23.67	
50+00 Treated	32.47	21.15
50+00 Untreated	21.02	
60+00 Treated	18.08	
60+00 Untreated	22.28	18.16

Table E-4. Moisture contents of in situ CBR locations and resilient modulus specimens.

Daviess KY 331		
Station	Moisture Content of In situ CBR Location (Percent)	Moisture Content of Resilient Modulus Specimen (Percent)
02+00 Untreated	17.14	
07+00 Treated	19.82	18.26
07+00 Untreated	19.61	
14+00 Treated	21.45	19.29
14+00 Untreated	18.26	
20+00 treated	19.70	14.25
20+00 Untreated	21.19	
32+00 Untreated	14.53	21.03
40+00 Untreated	20.03	

Table E-5. Moisture contents of in situ CBR locations and resilient modulus specimens.

Boone KY 842		
Station	Moisture Content of In situ CBR Location (Percent)	Moisture Content of Resilient Modulus Specimen (Percent)
10+00 Treated	26.40	
10+00 Untreated	22.01	
30+00 Treated	27.35	
30+00 Untreated	24.20	20.62
50+00 Treated	25.92	
50+00 Untreated	26.22	
70+00 Treated	23.08	
70+00 Untreated	24.71	
90+00 Treated	21.69	
90+00 Untreated	23.14	
110+00 Untreated	21.90	20.92

Table E-6. Moisture contents of in situ CBR locations and resilient modulus specimens.

Fayette US 25		
Station	Moisture Content of In situ CBR Location (Percent)	Moisture Content of Resilient Modulus Specimen (Percent)
-04+00 Ggeogrid	20.84	
10+00 Geogrid	18.59	
20+00 Treated	30.15	33.55
20+00 Untreated	33.05	24.14
30+00 Geogrid	07.16	
40+00 Treated	20.86	20.75
40+00 Untreated	24.20	20.31
50+00 Untreated	25.89	24.74
60+00 Treated	28.36	30.26
60+00 Untreated	25.36	25.62
70+00 Treated	23.02	28.44
70+00 Untreated	25.07	
77+15 DGA	03.14	
77+15 Treated	24.98	
77+15 Untreated	24.20	
81+75 Treated	22.38	
81+75Untreated	21.18	

Table E-7. Moisture contents of in situ CBR locations and resilient modulus specimens.

Hardin US 62		
Milepost	Moisture Content of In situ CBR Location (Percent)	Moisture Content of Resilient Modulus Specimen (Percent)
12.0 Treated	19.75	
12.0 Untreated	14.25	14.39
12.2 Treated	16.16	
12.2 Untreated	14.05	
12.45 Treated	18.45	
12.45 Untreated	15.77	
12.50 Treated	19.25	
12.50 Untreated	16.31	
12.80 Treated	15.06	
12.80 Untreated	15.14	12.31
12.90 Treated	19.33	23.72
12.90 Untreated	20.72	19.70
13.70 Treated	23.10	
13.70 Untreated	20.76	
13.75 EB Treated	19.22	20.19
13.75 EB Untreated	30.69	20.58
13.75 WB Treated	18.81	
13.95 EB Treated	21.03	
13.95 EB Untreated	15.38	16.27
13.95 WB Treated	18.73	
13.95 WB Untreat	21.05	21.18
14.2 Treated	19.06	23.45
14.2 Untreated	22.09	22.50
14.5 Untreated	13.63	15.25
14.6 Untreated	22.34	15.90

Table E-8. Moisture contents of in situ CBR locations and resilient modulus specimens.

McCreary US 27		
Station	Moisture Content of In situ CBR Location (Percent)	Moisture Content of Resilient Modulus Specimen (Percent)
655+75 6% Cement Treated	13.95	12.12
655+75 Untreated	11.56	
679+00 6% Cement Treated	13.62	14.05
679+00 Untreated	14.87	
733+40 Untreated Shale	7.32	
756+30 4% Cement	16.89	
756+30 Untreated	17.75	17.25
774+20 Cement	14.05	11.38
774+20 Untreated	13.59	
789+00 4% Cement	13.70	17.89
789+00 Untreated	9.51	9.97

Table E-9. Moisture contents of in situ CBR locations and resilient modulus specimens.

Hickman US 51		
Station	Moisture Content of In situ CBR Location (Percent)	Moisture Content of Resilient Modulus Specimen (Percent)
15+00 Treated	15.85	
19+25 Treated	20.34	
19+25 Untreated	16.26	
22+50 Treated	22.86	20.66
22+50 Untreated	18.93	14.71
30+00 Treated	18.57	
30+00 Untreated	20.42	17.26
40+00 Treated	20.20	
40+00 Untreated	17.42	
50+00 Treated	21.32	
50+00 Untreated	17.91	

Table E-10. Moisture contents of in situ CBR locations and resilient modulus specimens.

Owen US 127		
Station	Moisture Content of In situ CBR Location (Percent)	Moisture Content of Resilient Modulus Specimen (Percent)
0+00 Treated	23.61	27.06
0+00 Untreated	23.94	21.30
10+00 Treated	22.53	
20+00 Treated	17.96	14.98
20+00 Untreated	20.86	
30+00 Treated	21.61	
30+00 Untreated	25.10	
40+00 Treated	21.72	
40+00 Untreated	26.03	
50+00 Treated	23.91	
50+00 Untreated	23.87	20.87

Table E-11. Moisture contents of in situ CBR locations and resilient modulus specimens.

McCracken US 62		
Station / Stabilization	Moisture Content of In situ CBR Location (Percent)	Moisture Content of Resilient Modulus Specimen (Percent)
0+00 Bank Gravel	7.32	17.22
10+00 Bank Gravel	11.08	
20+00 Bank Gravel	9.30	
20+00 MKD Treated	24.94	28.03
30+00 MKD Treated	14.31	
40+00 Bank Gravel	8.31	
50+00 Bank Gravel	7.08	
50+00 MKD Treated	13.11	
60+00 Bank Gravel	10.09	
60+00 MKD Treated	16.93	

Table E-12. Moisture contents of in situ CBR locations and resilient modulus specimens.

Boyle US 127		
Station	Moisture Content of In situ CBR Location (Percent)	Moisture Content of Resilient Modulus Specimen (Percent)
14+00 Treated	27.16	26.73
14+00 Untreated	30.05	
25+00 Treated	27.50	24.24
25+00 Untreated	29.19	
50+00 Treated	28.22	
50+00 Untreated	31.04	23.27
60+00 Treated	24.68	24.98
60+00 Untreated	32.24	
75+00 Treated	26.97	21.02
75+00 Untreated	21.30	22.38
100+00 Untreated	14.37	

Table E-13. Moisture contents of in situ CBR locations and resilient modulus specimens.

Breckenridge US 60		
Station	Moisture Content of In situ CBR Location (Percent)	Moisture Content of Resilient Modulus Specimen (Percent)
10+00 Treated		11.06
10+00 Untreated	17.07	16.93
14+00 Treated	21.45	
14+00 Untreated	18.26	
40+00 Untreated	12.44	
60+00 Treated	11.13	
70+00 Untreated	18.33	16.73
120+00 Treated	18.69	
120+00 Untreated	16.38	
160+00 Treated	15.34	14.05
160+00 Untreated	15.98	
200+00 Untreated	15.59	

Table E-14. Moisture contents of in situ CBR locations and resilient modulus specimens.

Lee KY 11 Various		
Milepost / Stabilization Type	Moisture Content of In situ CBR Location (Percent)	Moisture Content of Resilient Modulus Specimen (Percent)
10.0 AFBC Treated	24.44	
10.0 Untreated	23.26	
10.2 AFBC Treated	30.47	
10.2 Untreated	19.98	
10.4 10% Soil Cement Treated	14.98	14.24
10.4 Untreated	19.98	
10.5 10 % Soil Cement Treated	19.12	15.70
10.5 Untreated	21.94	14.10
11.0 10 % Lime Treated	18.60	16.32
11.0 Untreated	20.95	
11.2 10% Lime Treated	15.09	20.42
11.2 Untreated	17.01	
12.0 MKD Treated	16.04	12.48
12.0 Untreated	18.25	17.45
12.5 MKD Treated	15.05	
12.5 Untreated	19.93	19.76
13.7 7% Soil Cement Treated	15.03	
13.7 Untreated	17.51	
14.1 Untreated	15.84	
14.5 AFBC Treated	26.33	19.39
14.5 Untreated	22.30	16.67
14.7 AFBC Treated	21.53	
14.7 Untreated	15.04	

Appendix F

Regression Plane Coefficients, k_1, k_2, and k_3, obtained from Models 4 and 5, Equations 11 and 12.

Table F-1. Regression plane coefficients (k_1, k_2, and k_3) of Models 4 and 5—Soil -Hydrated Lime Field Specimens

	Model 4					Model 5				Location
Sample ID	k_1	k_2	k_3	R^2		k_1	k_2	k_3	R^2	
Ander US127-1	1375.258	0.630933	-0.12859	0.923381		2728.072	0.519456	0.052416	0.863985	*75+38*
Anderson S-2	1384.958	0.852009	-0.29493	0.978908		3724.363	0.707908	-0.07321	0.951989	*U.S.127 30+00*
Boyle 127-1	1733.995	0.567501	-0.08407	0.929626		3079.813	0.488149	0.081268	0.944077	*14+00 by-pass*
Boyle 127-2	857.6512	0.841397	-0.22316	0.964382		2176.345	0.716113	-0.00358	0.960931	*75+00 by-pass*
Boyle 127-3	3763	0.587911	-0.2052	0.948228		7591.195	0.504997	-0.07703	0.989761	*60+00*
Boyle 127-6	7122.415	0.345795	-0.13228	0.974956		10876.5	0.285384	-0.05185	0.951422	*25+00*
Fayette US 25-2	3273.76	0.739041	-0.38927	0.979714		8109.107	0.639965	-0.23804	0.993751	*70+00 rt. cl.*
Fayette US 25-3	1978.966	0.717126	-0.29375	0.974267		4587.383	0.605053	-0.11973	0.964316	*20+00 lt. cl.*
Fayette US 25-5	5625.118	0.47428	-0.28516	0.949364		10383.68	0.408781	-0.19645	0.965826	*70+00 rt. cl.*
Fayette US 25-6	1260.515	0.777127	-0.31221	0.971196		3052.654	0.665888	-0.11734	0.97947	*40+00 LT.CL.*
Fayette US 25-8	1444.496	0.764931	-0.26174	0.974731		3399.69	0.646251	-0.05755	0.992547	*60+00 lt cl*
Hardin 62-2	2766.147	0.775589	-0.25276	0.991515		6883.674	0.634904	-0.04996	0.966022	*mp 12.90*
Hardin 62-3	3491.647	1.123906	-0.31205	0.933649		12856.83	0.950851	-0.03399	0.942157	*20+00*
Hardin 62-1	2602.185	0.373232	0.048404	0.93703		3616.32	0.305634	0.187188	0.934287	*0+00 13.75*
Lee KY 11-2	9362.763	-0.22695	0.71272	0.940177		4455.32	-0.15144	0.811185	0.912243	*M.P.11.2*
Lee KY11-6	19683.79	-0.16077	0.331825	0.965152		13259.83	-0.12987	0.367743	0.975888	*MP 11.0*
Owen 127-1	8689.741	0.427587	-0.20492	0.980877		14914.9	0.357093	-0.11347	0.980546	*0+00*
Owen-2	1414.089	0.74772	-0.21701	0.975203		3217.2	0.632114	-0.01247	0.977287	*20+00*
Shelby KY55	1474.369	0.740143	-0.33452	0.977562		3602.051	0.61931	-0.15939	0.979044	*30+75 North*
Shelby 55-1	12589.16	0.291089	-0.05761	0.863291		17654.25	0.248824	0.010086	0.904399	*50+00*
Shelby KY55-3	1482.247	0.801596	-0.37078	0.954666		3785.514	0.69446	-0.18621	0.970245	*20+00 rt.cl*
Shelby Ky 55-4	2696.773	0.62045	-0.42125	0.975564		6265.962	0.527654	-0.31898	0.986328	*40+00 lt cl*
Shelby KY55-5	784.2465	0.979469	-0.47335	0.943481		2533.291	0.81765	-0.23893	0.907416	*50+00 rt cl*
Shelby KY55-7	1542.47	0.21823	0.054181	0.920398		1799.373	0.176981	0.151695	0.895586	*30+00 rt.cl*
Trigg 27-1	18608.21	0.532874	-0.09136	0.90538		35660.78	0.422129	0.052224	0.976055	*600+00 WB*
Trigg 68-2	1388.172	1.303099	-0.2593	0.935758		6109.804	1.046473	0.118009	0.947203	*900+00*

Table F-2. Regression plane coefficients (k_1, k_2, and k_3) of Models 4 and 5—Soil –Cement, Lime Kiln Dust, and AFBC (Atmospheric Fluidized Bed Combustion Ash) Field Specimens

Cement treated subgrade core sample									
	Model 4				**Model 5**				
Sample ID	**k_1**	**k_2**	**k_3**	**R^2**	**k_1**	**k_2**	**k_3**	**R^2**	***Location***
Breckinridge 60-1	6965.86	-0.23998	0.542552	0.96628	3929.569	-0.20665	0.594357	0.980558	*160+00*
Breckinridge 60-2	1701.984	1.310212	-0.26486	0.967891	7056.693	1.112724	0.100999	0.968878	*10+00*
Breckinridge 60-3	11989.14	0.859286	-0.30419	0.941175	36979.48	0.685254	-0.11363	0.94007	*160+00*
Daviess 331-4	14872.75	0.65925	-0.13973	0.897789	33789.26	0.516004	0.047136	0.936458	*14+00 MP 1.0*
Daviess 331-5	5115.954	0.999746	-0.22244	0.959842	17263.16	0.759852	0.081208	0.946832	*20+00*
Daviess 331	25669.79	0.341658	-0.06192	0.758505	37959.44	0.293468	0.019807	0.843362	*7+00*
Lee KY 11-1	10809.03	-0.12546	0.550848	0.970464	6642.096	-0.10588	0.648579	0.980912	*MP 13.70*
Lee KY 11-4	10353.98	-0.03843	0.330418	0.960637	8080.876	-0.03753	0.400456	0.975426	*MP 10.4*
Lee KY 11-5	9382.974	-0.20171	0.510998	0.944957	5685.013	-0.18804	0.571057	0.96774	*MP 10.5*
McCreary 27-2	270.7807	1.458174	-0.10564	0.921579	986.4915	1.272888	0.395074	0.945368	*679+00*
McCreary 27-1	16529.78	0.627584	-0.09449	0.91894	36460.82	0.480953	0.075888	0.954697	*774+00*
McCreary 27-3	35582.44	0.293867	-0.00636	0.901652	47406.3	0.251831	0.085465	0.921392	*655+75*
MKD treated									
Hickman US 51-1	1753.133	0.589592	-0.23297	0.974781	3496.406	0.496498	-0.0882	0.95663	*2+25*
Lee KY 11-3	11542.86	0.043069	0.267912	0.936752	10298.95	0.024678	0.3472	0.952401	*MP 12.0*
McCracken 62-1	1682.379	0.627755	-0.23745	0.949596	3406.653	0.533957	-0.0751	0.957567	*20+00*
AFBC Treated subgrade									
LEE KY 11-7	2179.389	0.701728	-0.40833	0.984026	5197.786	0.594927	-0.25374	0.971114	*MP14+50*

Table F-3. Regression plane coefficients (k_1, k_2, and k_3) of Models 4 and 5—untreated field subgrade specimens.

Untreated Field Subgrade Specimens										
	Model 4					Model 5				
Sample ID	k_1	k_2	k_3	R^2		k_1	k_2	k_3	R^2	*Location*
Anderson 127-3	6879.192	0.550719	-0.25726	0.943197		13738.97	0.463986	-0.13909	0.94499	*30+00*
Anderson 127-4	1738.063	0.628647	-0.21941	0.940774		3622.404	0.536005	-0.07305	0.951449	*50+00*
Anderson 127-5	10392.7	0.314773	-0.19875	0.887694		16063.83	0.268672	-0.15106	0.92582	*50+00*
Anderson 127-6	16894.8	0.160339	-0.16376	0.856795		22206.19	0.134851	-0.15768	0.882516	*11+55*
Boone 842-1	7181.008	0.321883	-0.30354	0.893168		11810.76	0.287728	-0.28301	0.948852	*30+00*
Boone 842-2	2812.511	0.530662	-0.40198	0.95649		5848.779	0.461379	-0.32099	0.987502	*110+00*
Boyle 127-4	12101.26	0.28286	-0.27943	0.951226		19268.48	0.239627	-0.26372	0.972028	*75+00*
Boyle 127-5	890.4185	0.928164	-0.20662	0.967953		2515.465	0.782793	0.037523	0.972791	*50+00*
Breckinridge 60-7	8939.437	0.359252	-0.09126	0.924037		14041.9	0.284801	-0.00413	0.934449	*70+00*
Breckinridge 60-8	1672.632	0.681224	-0.25613	0.912878		3703.885	0.595971	-0.10788	0.956138	*10+00*
Breckinridge 60-9	5272.287	0.451019	-0.16758	0.956874		9191.073	0.386578	-0.07594	0.988705	*70+00*
Daviess 331-2	3930.104	0.657064	-0.25153	0.990462		8669.56	0.537791	-0.08719	0.952633	*7+00*
Daviess 331-3	8161.916	0.357106	-0.28758	0.98264		13810.18	0.299853	-0.24075	0.985294	*32+00*
FayetteUS25-10	8003.028	0.520534	-0.30286	0.897792		15714.41	0.438378	-0.1966	0.938694	*60+00 LT. CL.*
Fayette US 25-4	11523.13	0.354335	-0.29994	0.881732		19173.17	0.309826	-0.2573	0.919422	*20+00 LT. CL.*
Fayette US 25-7	11469.42	0.298611	-0.22936	0.877805		17465.08	0.255938	-0.18516	0.907852	*40+00 LT. CL.*
Fayette US 25-9	8876.106	0.440926	-0.30671	0.927376		16211.28	0.370554	-0.22776	0.953287	*60+00 LT. CL.*
Hardin 62-11	8726.631	0.418853	-0.17839	0.948928		15136.2	0.354723	-0.10591	0.977292	*MP 12.80*
Hardin 62-12	9264.997	0.313165	-0.18187	0.952536		14439.49	0.261847	-0.13569	0.976171	*10+00*
Hardin 62-4	4316.603	0.381195	-0.42046	0.935129		8397.681	0.310691	-0.40645	0.939541	*12.9*
Hardin 62-5	7414.914	0.37116	-0.25682	0.936606		12306.9	0.31994	-0.19706	0.975651	*20+00*
Hardin 62-6	9547.053	0.298103	-0.33415	0.985011		15701.29	0.250587	-0.31687	0.991451	*41+50*
Hardin 62-7	7707.155	0.318727	-0.27167	0.950003		12518.22	0.273393	-0.2402	0.970471	*10+00*
Hardin 62-8	7444.591	0.257056	-0.44266	0.959598		12689.73	0.221779	-0.47221	0.974105	*0+00*
Hardin 62-9	3994.746	0.546871	-0.21989	0.963898		8102.921	0.455013	-0.11257	0.984663	*MP 12.00*

Table F-4. Regression plane coefficients (k_1, k_2, and k_3) of Models 4 and 5—untreated field subgrade specimens

Untreated Field Subgrade Specimens										
Model 4						Model 5				
Sample ID	**k_1**	**k_2**	**k_3**	**R^2**		**k_1**	**k_2**	**k_3**	**R^2**	***Location***
Hickman 51-3	1744.756	0.696225	-0.31286	0.968208		3901.656	0.6046	-0.14676	0.985949	*2+25*
Hickman 51-2	2959.609	0.592218	-0.22452	0.962938		6187.246	0.488849	-0.09045	0.964005	*30+00*
Lee KY 11-10	4435.203	0.426409	-0.39342	0.96598		8564.029	0.369758	-0.35624	0.98345	*MP 12.50*
Lee KY 11-9	1913.003	0.611591	-0.37849	0.972307		4314.619	0.512594	-0.26297	0.959659	*MP 14.50*
McCreary 27-5	11309.27	0.319631	-0.40818	0.978825		19989.49	0.269111	-0.40401	0.981953	*756+30*
Owen 127-	5603.878	0.376279	-0.36556	0.963032		9995.923	0.327528	-0.33235	0.990365	*50+00*
Owen 127-3	2053.581	0.553019	-0.28922	0.943557		4030.346	0.484831	-0.17647	0.980241	*0+00 North*
Owen 127-5	9201.979	0.266373	-0.33738	0.932238		14636.67	0.236855	-0.33875	0.971087	*50+00*
Shelby KY55-10	13575.83	0.402995	-0.21658	0.909384		22121.62	0.348514	-0.12863	0.945585	*40+00 lt. cl.*
Shelby Ky55 2	21739.61	0.177473	-0.14084	0.879469		28001.33	0.155402	-0.11916	0.917678	*10+00 North rt.*
Shelby 55-2	7215.307	0.36721	-0.10256	0.939137		11276.48	0.310841	-0.02419	0.985906	*60+00*
Shelby KY55-6	2928.95	0.425309	-0.09359	0.928454		4534.742	0.368871	0.025083	0.940385	*60+00 lt.cl*
Shelby KY55-8	9068.724	0.281535	-0.49536	0.957346		15886.72	0.251444	-0.52334	0.976914	*30+00 rt cl*
Shelby KY55-9	15459.56	0.370126	-0.19161	0.90976		24107.58	0.318614	-0.10791	0.935674	*20+00 lt. cl.*
Trigg 68-3	3657.619	0.622114	-0.28341	0.90086		8174.689	0.516383	-0.15481	0.906476	*900+00*
Trigg 68-4	5405.929	0.620265	-0.32146	0.949323		12294.4	0.495551	-0.18187	0.926336	